ON ENCASED BRICK COLUMNS

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by

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to the

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MARCH, 1989

To my

Parents and Guide

V.90

CE-1989-M-PAT-ENC

CERTIFICATE

This is to certify that the work presented in this thesis entitled On Encased Brick Columns, by Prithvi Pati has been carried out under my supervision and it has not been submitted elsewhere for a degree.

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LIST OF SYMBOLS AND ABBREVIATIONS

Symbol:

U	=	Released energy during crack propagation.
σ	=	Applied stress.
a	=	Crack length
E	=	Young's modulus of elasticity.
P	=	Vertical compressive load.
P_u	=	Ultimate compressive load at failure.
σ_v	=	Vertical stress.
σ_{bt}	=	Tensile stress in brick.
σ_{mc}	=	Compressive stress in mortar.
E_b	=	Elastic modulus of brick.
E_m	=	Elastic modulus of mortar.
B	=	Lateral dimension of square column.
h	=	Depth of brick.
t	=	Thickness of mortar joint.
ε_{tb}	=	Tensile strain in brick.
ε_u	=	Ultimate tensile strain in brick at failure.

Abbreviation:

IS	Indian Standard Code of Practice
ACI	American Concrete Institute
ABS	American Bureau of Shipping
LD	Location of Dial Gauge

ABSTRACT

The aim of the present study is to compute the load carrying capacity of brick columns with and without ferrocement encasement. An attempt has been made to compute the theoretical load capacity values, which are compared with the experimental ones. The columns are loaded axially as well as eccentrically. The initiation of cracks and the modes of failure have been recorded. The failure of encased brick columns is found to be different from those of reinforced concrete ones. One of the major conclusions of study is that a rational theoretical model must be developed.

CHAPTER I

INTRODUCTION, LITERATURE SURVEY AND REVIEW

1.1 GENERAL

Rural development is a process whereby changes are introduced in the rural environment to bring about sustained improvements on the quality of life among the rural people. The importance of rural development to developing countries is fundamental as the vast majority (about 90%) of its population lives in rural areas with in the grip of poverty [1]. About 70% of the world population is in developing countries [2].

The rural population continues to be faced with problems, poor housing, water storage, inadequate sanitation and other basic needs. The basic causes of the problems are the accelerated population increase, rural to urban migration and low income.

Solution to the problems lies in integrated rural development with proper application of science and technology. Rapid introduction of technology which is both capital saving and labour intensive, will be a contributing factor to a successful solution of the problems.

To overcome the housing problem and to provide earthquake and storm resistant low cast housing, the housing technology for developing countries must take into account the following factors.

(a) Simple design in keeping with local culture.

(b) Use of readily available materials.

(c) Ease of fabrication geared to an unskilled labour force.

(d) Great flexibility in the type of structure.

(e) Adaptability to site location and climatic conditions.

The adoption of ferrocement as an appropriate construction material for housing as well as for several other structures makes it outstanding and unique. The greatest factors leading to the acceptance of ferrocement are: the availability of its raw materials in most of the developing countries, it can be fabricated into any shape and adopted to environmental and traditional customs of the country; no skilled labour is required and it is suitable for construction on self help basis resulting in a feeling of achievement and involvement among the people.

Thus, there is a vast potential for ferrocement in the developing countries. It is a highly versatile construction material satisfying the above criteria and at the same time yields significant economies during construction [3].

About 40% of Indian population is below poverty line but consumption of natural resources is at such a level that the cost of essential building commodities is reaching sky high. So, there is a need for low cost houses for the poor and the needy. For the construction of rural houses, villagers largely depend on local building materials. In various parts of India brick work is extensively used as bricks are available there as readily

available local material. For example the quality of bricks along Gangetic plain is very good and are extensively used. With proper precautions brick work, probably more than any other building material can function satisfactorily with the minimum of maintenance and cost. Its inherent advantages, in particular high resistance to fire and corrosive chemicals make it one of the most popular materials used in modern buildings [4].

1.2 FERROCEMENT

Although ferrocement has been used as a material of construction for various applications in many countries, only recently there has been a growing interest even in the western hemisphere to use it as a structural material. The distribution in concrete matrix of continuous fine reinforcement compared to conventional larger size bars give ferrocement several advantages which were recognized for quite some time in many developing countries without the support of vigorous analysis.

Based on the past experience the following topics are discussed here:

(i) Basic definition of ferrocement.

(ii) Useful structural properties.

(iii) The concept of concrete composites.

(iv) General characteristics of continuous arranged micro-reinforcement.

(v) Applications.

1.2.1 Definition of Ferrocement

As the material "Ferrocement' was used for a long time in boat building and similar allied structures rather than in structural applications, according to the American Bureau of shipping it is:

"A thin highly reinforced shell of concrete" in which the steel reinforcement is distributed widely throughout the concrete, so that the material under stress acts approximately as a homogeneous material. The strength properties of material are to be determined by testing a significant number of samples. The underlined portion of above definition may have different meanings of ferrocement to different people [5]. Bezukladov [6] defined it in terms of the ratio of the surface area of reinforcement to the volume of the composite. In this manner, ferrocement is separated from the conventional reinforced concrete somewhat arbitrarily, he assigned the specific surface greater than 2 cm^2/cm^3 to ferrocement which then behaves more or less as a homogeneous material. Less than 2 cm^2/cm^3 is considered reinforced concrete.

Shah [7] called it a composite made with mortar and a fine diameter continuous mesh as reinforcement, which has higher bond due to its smaller size and a larger surface area per unit volume of mortar. Accordingly, this ratio may be as much as ten times that which is observed in conventional reinforced concrete; this results in failure of ferrocement in tension by the actual breaking of wire mesh and a much higher cracking strength in the matrix.

As a composite, certain characteristics of ferrocement may thus be summarized as follows:

(a) Since the wire mesh (reinforcement) is much stronger in tension compared to the matrix (mortar), the role of the matrix is to properly hold the mesh in place to give a proper protection and to transfer stresses by means of adequate bond.

(b) Compressive strength of this composite is generally a function of the matrix compressive strength, while the tensile strength is a function of the mesh content and its properties.

(c) It follows from (b) above that the stress-strain relationship of ferrocement in tension may show either a complete elastic behaviour (upto fracture of reinforcing mesh) or some inelasticity depending upon the yielding properties of the mesh.

(d) Since the properties of this composite are very much a function of orientation of the reinforcement, the material is generally anisotropic and may be treated as such in the theoretical analysis.

The above discussion indicates the variety of approaches that have been made in a structural definition of ferrocement. The American Concrete Institute finally adopted the following definition.

"Ferrocement is a type of thin wall reinforced concrete construction, where usually a hydraulic cement is reinforced with

layers of continuous and relatively small diameter mesh. The mesh may be made of metallic materials or other suitable materials".

1.2.2 Properties as Building Material

Structural properties of ferrocement differ from reinforced cement concrete mainly due to the fine dispersion of reinforcement in ferrocement. The important parameters associated with the behaviour of ferrocement are as follows.

(a) Specific Surface:

Specific surface is defined as the actual surface area of reinforcement present per unit volume of mortar.

(b) Volume Fraction:

It is a fraction of the total volume of the specimen which is occupied by reinforcement.

Following are the main properties of ferrocement studied so far.

(c) Tensile Behaviour and Crack Propagation:

Unlike reinforced concrete tensile behaviour of ferrocement is considerably different. This is mainly because the reinforcement is spaced closer and uniformly than in reinforced concrete and its smaller diameter results in a larger specific area. This in turn affects cracking behaviour (finer and more number of cracks) in ferrocement.

Tensile behaviour is characterised by cracking (first crack) toughness elongation and ultimate strength of specimen. The work

of Naaman and Shah [8] has indicated that the stress level at which the first crack appeared and the crack spacing and number of cracks at failure were a function of the specific surface of reinforcement as indicated in Fig. 1.1.

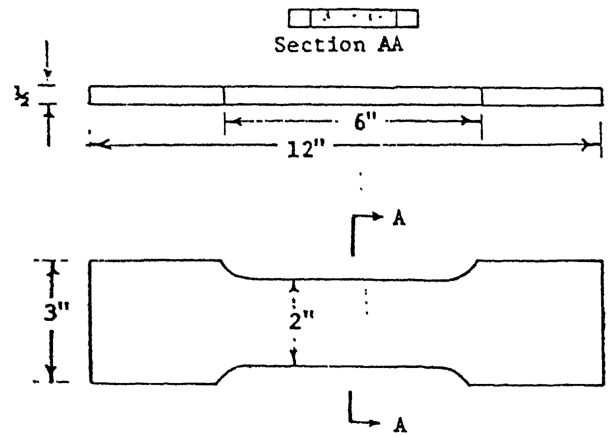


Fig. 1.1a: Tensile specimen.

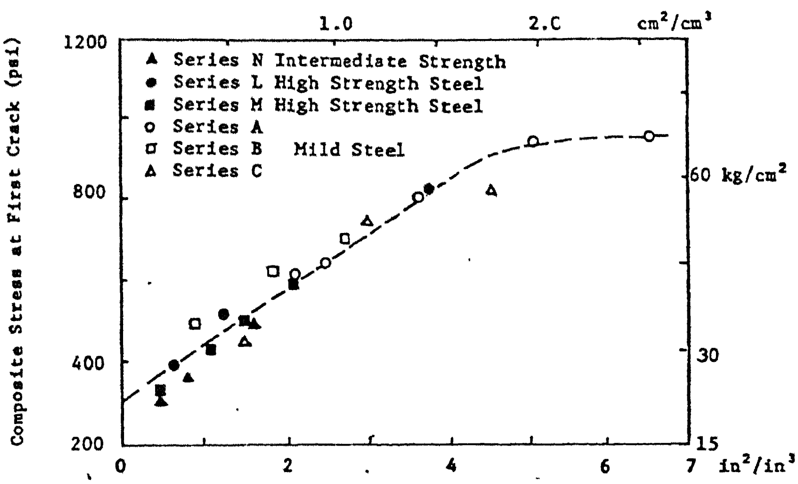


Fig. 1.1b: Stress at first crack vs. specific surface reinforcement.

Toughness provided by a single layer was increased by increasing the number of layers of reinforcement. The ultimate load carrying capacity of ferrocement was the same as the load carrying capacity of the reinforcement in that direction. This should be expected since the load is carried by the reinforcement itself after the mortar has cracked. Thus tensile strength is largely governed by the amount of wire mesh reinforcement and is independent of the strength and composition of mortar.

Reinforced cement concrete is the most extensively used building material in the world now-a-days. In the materials with small internal flaws or microcracks, under tensile loading, unstable crack propagation takes place.

According to Griffith [9]

$$U = f(\sigma, a, E) \tag{1.1}$$

where,

U = Released energy during crack propagation.

σ = Applied stress.

a = Crack length

E = Young's modulus of elasticity

According to Griffith's hypothesis crack will begin to propagate unstably if the rate of elastic energy release is equal to or greater than the energy required to extend the crack. It is clear that the resistance of the materials with a particular specific surface, energy can be increased by keeping the applied stress low, reducing the flaw length or increasing the modulus of

elasticity, E. But it is not easy to manipulate elastic modulus (E) and keeping applied stress (σ) low. Romualdi and Mandel [10] have shown that the strain energy release during crack propagation is reduced due to the shear bond between the wire and the mortar, resulting in arrest of cracks.

Generally, the distributed reinforcement checks the crack length and at the same time generates a large number of microcracks indicating that ferrocement behaves very much like a homogeneous material in contrast with reinforced concrete.

(d) Flexural Strength (Behaviour in Bending):

It is interesting to note that flexural strength of a ferrocement beam can be calculated using the conventional method of analysis of reinforced concrete. The load deflection relations shown in Fig. 1.2. clearly indicate this.

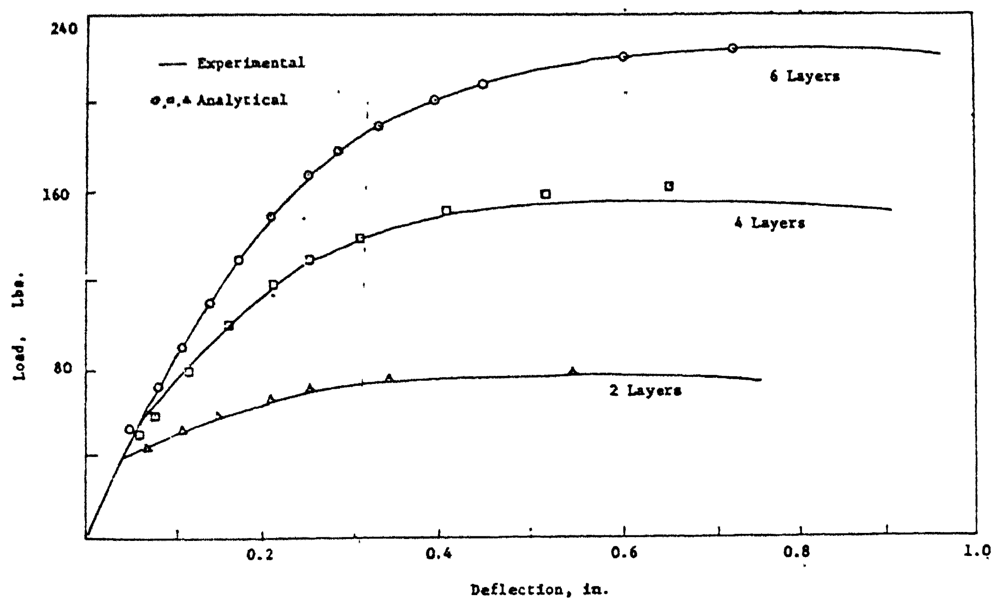


Fig. 1.2: Comparison of experimental and analytical load-deflection curves for ferrocement specimens

The figure also shows that increasing the amount of

reinforcement does not significantly increase the cracking moment of the beam, indicating that cracking behaviour is different from the cracking behaviour in tension. It should also be noted that there is no sudden failure as might be expected in the "over reinforced" concrete beam. Needless to say that compared to an average reinforced concrete beam (which is generally under reinforced), the ferrocement beam due to several layers of wire mesh tends to be over reinforced. It is therefore important to insure that indeed ferrocement will not fail similarly to an over-reinforced concrete beam. Analytical and experimental evaluations are reported by Johnston and Mowat [11]. Logan and Shah [12] Balaguru et.al. [13] and Pama et.al. [14]. Rao and Gowdar [15] tested ferrocement under central line load and Logan and Shah tested ferrocement beams in flexure by applying third point loading. Rajagopalan and Parameshwaran [16] have used a theoretical approach to predict ultimate moment and crack behaviour.

Lee, Pama and Raisinghani [17] and Raisinghani [18] tested slab elements. They have observed that:

(i) The flexural tensile stress increased with increase in specific surface. The moment curvature relation is trilinear which is a good approximation of the actual one.

(ii) Poisson's ratio and modulus of rigidity increase with increase in specific surface.

(iii) Till the tensile crack is developed, the response is elastic. This is followed by a strain hardening stage where increasing number of cracks are seen with increasing stress. The increase in strain is mainly due to increasing crack width. For crack width more than 0.5 mm the material begins to flow. The maximum permissible crack width is 0.3 mm. After material begins to flow it eventually collapses. At this stage, the deflection is governed by the crack width alone.

(e) Impact:

Impact strength may be looked upon as special property. It is a useful parameter in applications related to off-shore structures. Impact strength is defined as the energy absorbed by the specimen when struck by a swinging pendulum dropped from a constant height. The damage is measured by the relative flow of water through the damaged surface for a fixed energy absorbed. Shah and Key [19] found that higher the specific surface and strength of the meshes the lower is the damage due to impact loading.

It has been predicted that the effect of impact loads on ferrocement is localised and failure is characterised by a widely dispersed area of shattered mortar and even under such conditions, the material remains impervious. Resistance to impact is one of the assets of ferrocement. The material has high toughness when compared with reinforced cement concrete.

(f) Fatigue:

Fatigue strength of ferrocement may also be looked upon as special property. This strength may play an important role in restricting the use of ferrocement in structures subjected to such a loading as in bridges.

It was observed by Raisinghani that flexural rigidity decreases and stress-strain relations become linear with increasing number of load cycles. Failure is usually sudden and brittle.

Ferrocement can withstand upto 2×10^6 cycles at stress levels of about 40% and 65% of the upper and lower yield loads respectively.

(g) Ductility and Durability:

Durability and ductility are much higher than those found in other forms of construction. The two dimensional continuity of fibres imparts excellent resistance to disintegration and facilitates repair of damaged zones simply by plastering.

These properties are achieved with a thickness that is generally less than 25 mm, a dimension which is almost impossible with other forms. High levels of performance in ductility, strength, cracking resistance and durability can be achieved even if the quality control is not upto the standard.

(h) Creep:

According to Raisinghani creep rates are inversely

proportional to curing time and for low stress levels, creep attains a steady rate in about 45 days.

(j) Water (Liquid) Retaining Capacity:

Another special property to be noted is that of water retention when application of ferrocement is considered in liquid storage tanks. The important aspect here is small crack width so that leakage may be minimal. Work by Shah and Naaman [20], indicated that crack widths in ferrocement for the same steel stress are smaller than in reinforced concrete by one order of magnitude. This makes it a better choice on materials for water retaining structures.

(k) Greater Resistance to Environmental Loads:

Because of one piece construction, it offers greater resistance to environmental loads.

(l) Ability to Adopt Curvature of Construction Unit:

It has the ability to adopt the curvature of the units to be constructed.

(m) Easy to Place and Less Form Work Requirement:

It is easy to place and can be cast with little or no formwork.

1.3 BRICK WORK

1.3.1 Brick Mortar and Brickwork:

Various investigators [21,22,23,24,25] have conducted experiments on bricks, mortar and brickwork. On the basis of experiments conducted so far they have concluded that:

i) Edge wise and end wise crushing strength of bricks are 47% and 43% of flatwise crushing strength respectively.

ii) Tensile strength of brick is 30% to 40% of transverse strength (modulus of rupture) and shear strength is in the range of 30% to 45% of the net compressive strength.

iii) Poisson's ratio of brick falls in the range of 0.05 to 0.10.

iv) Modulus of rupture varies with the square root of the crushing strength.

v) The tensile bond strength of the brick mortar joint interface does not vary with mortar compressive strength but its rate of increment damps down with age.

vi) The absorption capacity of brick is the predominant factor in determining the bond strength between brick and mortar. It has also been found that mortars containing the maximum amount of water consistent with workability have higher brick mortar bond strength than mortar having less water.

vii) There is no consistent relationship between mortar strength and tensile bond strength but bond strength increases as mortar flow increases.

viii) Maximum water addition to mortar will improve the tensile bond characteristics while reducing compressive strength durability and volume change.

ix) Mortars having high compressive strength and low air content produce relatively high bond strength between brick and mortar. It was also found that grout prepared by mixing water and ingredients to get desired flow is having lower air content compared to the grout prepared from a mortar of required mix by adding additional amount of water.

x) Moist curing of masonry prolongs the hydration period and appreciably increases the tensile bond of mortars to masonry units.

xi) The water for curing should be optimum, if provided in excess, it may saturate the brick resulting in decrease in adhesion between brick and mortar as well as the strength of the brick itself.

xii) The stress in brick can vary over a wide range, the strain at fracture always lies around 0.001. Putting it in another way, the ratio of stress at failure to Young's modulus is constant at approximately 0.001. This critical strain is very much less than would be expected from theoretical consideration of the behaviour of an ideal crystal structure. The Young's modulus of bricks ranges from 3.5 kN/mm^2 for low strength bricks upto 34 kN/mm^2 for high strength bricks. Although under a static load bricks show little or no plastic deformation prior to failure, yet under small alternating stresses remote from fracture bricks exhibit considerable plastic or irreversible strains inside the specimen.

xiii) The stress/strain relationship for mortar is generally a curve showing distinct plastic characteristics. In the absence of direct proportionality between stress and strain it is difficult to give definite values of Young's modulus of mortar and its strength.

1.3.2 Behaviour of Brickwork in Compression

Fig.1.3 shows the graph of brick strength against brick work strength tested at 28 days after casting. The results which came

from different sources were obtained by testing 0.229 m brickwork cubes which is one of the methods used for measuring brickwork

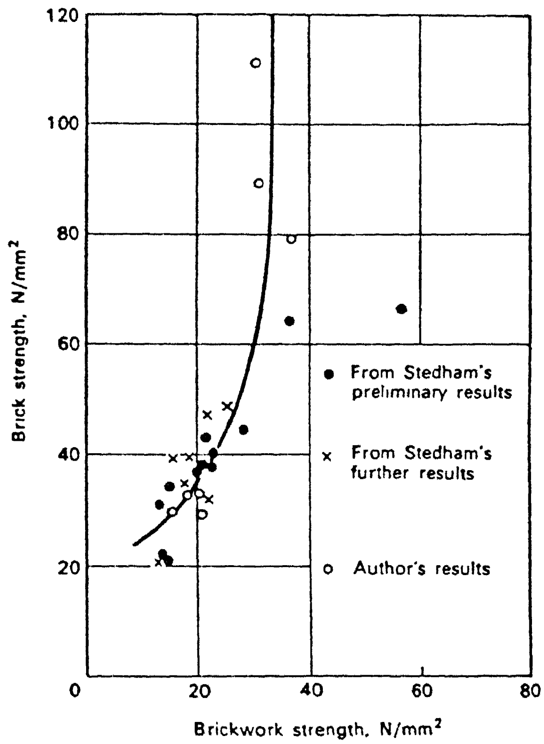


Fig. 1.3: Brick strength against brickwork cube strength.

strength. The graph is in a form of an exponential curve with a limiting brick work strength of approximately 35 MPa. There is very little increase in brick work strength above a brick strength of 83 MPa.

Fig.1.4 shows a graph of mortar strength and brick work cube strength both tested at 28 days, with brick strength kept constant. Once again the graph is a curve with the increase in brick work strength getting progressively smaller as the mortar strength increases. These and other tests on brick work cubes and

on full size walls have produced a number of empirical formulae relating brick mortar and brickwork strength. Most of these indicate that brickwork strength is proportional to the square root of brick strength and the fourth root of mortar strength.

The typical value of ratio, wall strength/cube strength is approximately 0.7.

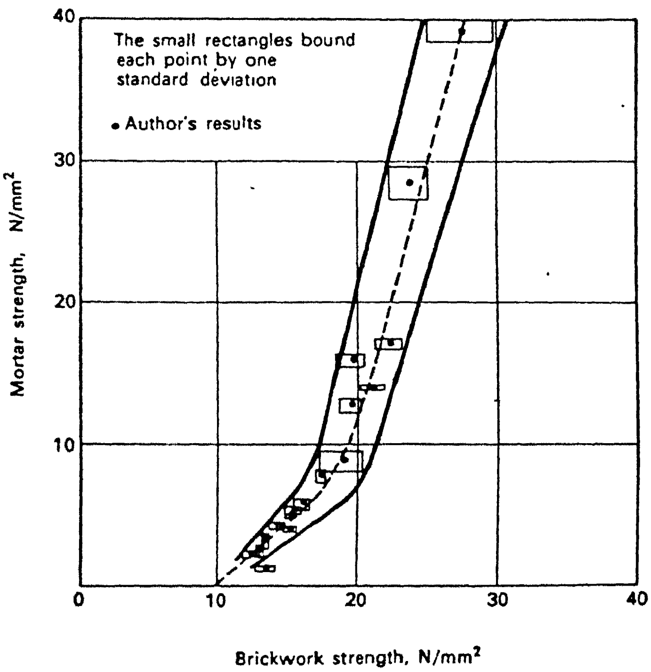


Fig. 1.4: Mortar strength against brickwork cube strength.

There are a number of useful indices which could be used to predict brickwork strength. One of these is the density of bricks.

Fig.1.5 shows a plot of brickwork cube strength against the dry density of bricks. The relationship is almost linear.

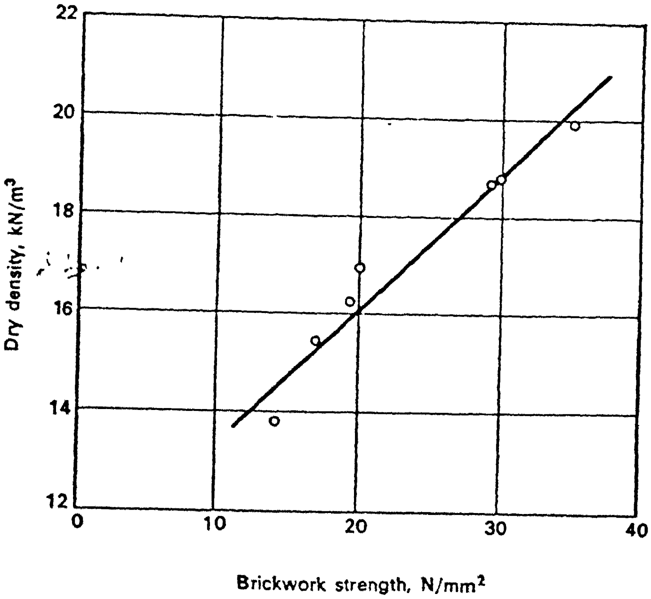


Fig. 1.5: Brickwork cube strength against dry density of bricks.

Another useful index is the dynamic modulus of mortar. It is seen in graph that there is again a linear relationship between brick work strength and dynamic modulus.

Tests on brickwork cubes showed that they are less sensitive to curing time than concrete or mortar. This is because a high proportion of brick work are bricks whose strength after firing remains constant. After 7 days brick work reaches approximately 80% of its ultimate strength and after 14 days, 95%. The standard time for testing brick work is 28 days when it has almost reached

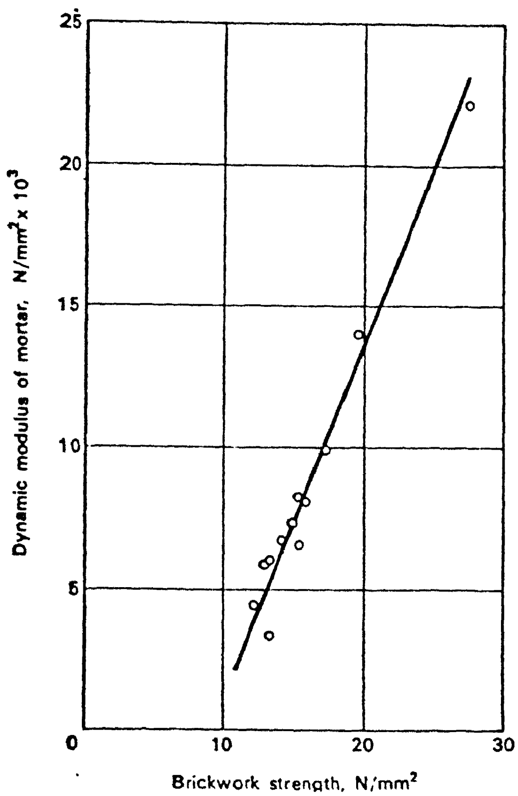


Fig. 1.6: Brickwork cube strength against the dynamic modulus of mortar.

its ultimate strength. The thickness of mortar bed joints has quite a significant effect on brick work strength. The strength of brick work decreases with increase in thickness of bed joints.

Excessive water suction by bricks from mortar also reduces brick work strength.

The effect of workmanship in strength of brick work is of paramount importance. It is well known that there are considerable practical difficulties in ensuring that frogs are completely filled with mortar. It has been shown by tests that when single frogged bricks are laid frog down, the resulting brick work was 20% weaker than when laid with frogs up.

1.3.3 Mechanism of Failure in Brick Work Under Axial Load

Failure in brick work under axial compression is normally by vertical splitting due to horizontall tension in the bricks. Fig. 1.7 shows a typical failure pattern in a brick work wall. The reason for this type of failure is due mainly to the widely different strain characteristics of the bricks and mortar joints. The mortar is less rigid than the brick and under load its tendency is to spread laterally to a greater extent than the brick.

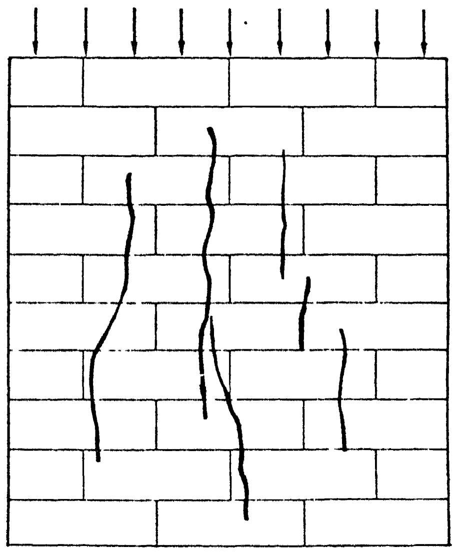


Fig. 1.7: Typical failure pattern in a brickwork wall.

Differential movement is prevented by the bond between the brick and mortar and consequently the mortar is put into a state of biaxial compression and the brick into biaxial tension [26].

CHAPTER II

THEORY OF FAILURE

2.1 SIMPLE THEORY OF FAILURE OF COLUMN UNDER AXIAL LOADING

It is assumed in the present theory that bricks as well as mortar remain elastic upto failure. Brick work under compressive loading fails when the tensile strain in the brick reaches its ultimate value. This fact can be used in evaluating bearing capacity of a column. A qualitative approach has been made to develop the theory.

Consider a brickwork assembly subjected to a vertical compressive load P. Let vertical stress due to this load be σ_v. Next consider a single brick which is bound at top and bottom by mortar joints of thickness t. Due to bonding between the brick and mortar, a composite action between the two is setup with both of them being strained together as one unit. The mortar is generally less rigid than the brick. In composite action the brick is subjected to biaxial tension and the mortar is subjected to biaxial compression. If the tensile stress developed in the brick is σ_{bt} and compressive stress developed in the mortar is σ_{mc}, then,

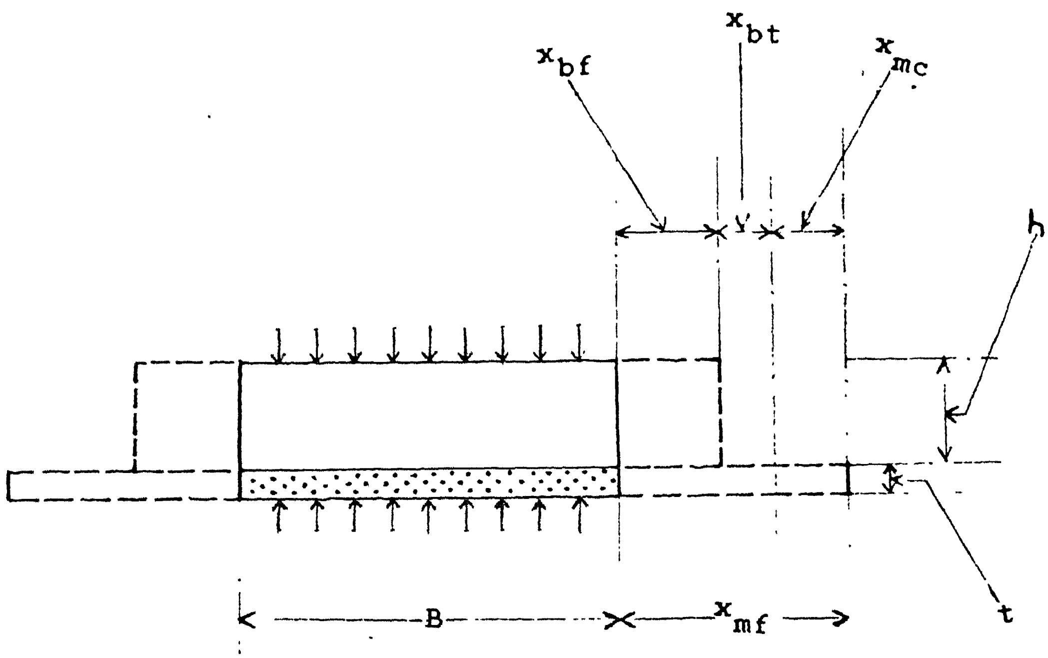


Fig. 2.1: Lateral expansion of brick and mortar under vertical compressive load.

Lateral strain in brick due to biaxial tension

$$= \frac{\sigma_{bt}}{E_b} (1 - \nu_b) \tag{2.1}$$

where E_b and ν_b are Elastic modulus and Poisson's ratio of brick respectively.

Lateral expansion of brick

= lateral strain x lateral dimension

$$= \frac{\sigma_{bt}}{E_b} (1 - \nu_b) B \tag{2.2}$$

Similarly lateral contraction of mortar due to biaxial compression:

$$= \frac{\sigma_{mc}}{E_m} (1 - \nu_m) B \tag{2.3}$$

where E_m and ν_m are Elastic modulus and Poisson's ratio of mortar respectively.

If there were no bond between brick and mortar, then,

$$\text{Free lateral expansion of brick} = \frac{\sigma_v}{E_b} \nu_b B \tag{2.4}$$

$$\text{and free lateral contraction of mortar} = \frac{\sigma_v}{E_m} \nu_m B \tag{2.5}$$

Fig. 2.1 shows diagrammatically the free lateral expansion of brick and mortar due to externally applied stress and the resultant expansion of the composite. From figure we see that,

Lateral contraction in mortar + Lateral expansions of brick

= Difference in free lateral expansions of mortar and brick

$$\text{or,} \quad \frac{\sigma_{bt}}{E_b} (1-\nu_b) B + \frac{\sigma_{mc}}{E_m} (1-\nu_m) B = \frac{\sigma_v}{E_m} \nu_m B - \frac{\sigma_v}{E_b} \nu_b B$$

$$\text{or,} \quad \frac{\sigma_{bt}}{E_b} (1-\nu_b) + \frac{\sigma_{mc}}{E_m} (1-\nu_m) = \frac{\sigma_v}{E_m} \nu_m - \frac{\sigma_v}{E_b} \nu_b \tag{2.6}$$

From equilibrium

Tensile force = Compressive force

$$\text{or} \quad \sigma_{bt}\, h\, B = \sigma_{mc}\, t\, B \tag{2.7}$$

From equations (2.6) and (2.7), we get,

$$\frac{\sigma_{bt}}{E_b}(1-\nu_b) + \frac{\sigma_b h}{E_m t}(1-\nu_m) = \frac{\sigma_v}{E_m}\nu_m - \frac{\sigma_v}{E_b}\nu_b$$

$$\text{or} \quad \sigma_{bt}\left[\frac{(1-\nu_b)}{E_b} + \frac{h}{E_m t}(1-\nu_m)\right] = \sigma_v\left(\frac{\nu_m}{E_m} - \frac{\nu_b}{E_b}\right)$$

$$\text{or} \quad \frac{\sigma_{bt}}{E_b}\left[(1-\nu_b) + \frac{h\ E_b}{t\ E_m}(1-\nu_m)\right] = \sigma_v\left(\frac{\nu_m}{E_m} - \frac{\nu_b}{E_b}\right)$$

$$\text{or} \quad \frac{\sigma_{bt}}{E_b} = \frac{\sigma_v\left(\frac{\nu_m}{E_m} - \frac{\nu_b}{E_b}\right)}{\left[(1-\nu_b) + \frac{h\ E_b}{t\ E_m}(1-\nu_m)\right]} \tag{2.8}$$

Total lateral tensile strain in brick:

$$\xi_{tb} = \frac{\sigma_v}{E_b}\nu_b + \frac{\sigma_{bt}}{E_b}(1-\nu_b) \tag{2.9}$$

Substituting for $\frac{\sigma_{bt}}{E_b}$ from equation (2.8) in equation (2.9),

$$\xi_{tb} = \frac{\sigma_v}{E_b}\nu_b + \frac{\sigma_v\left(\frac{\nu_m}{E_m} - \frac{\nu_b}{E_b}\right)(1-\nu_b)}{\left[(1-\nu_b) + \frac{h\ E_b}{t\ E_m}(1-\nu_m)\right]} \qquad \text{(2.10}$$

$$\text{or,} \quad \xi_{tb} = \sigma_v\left[\frac{\nu_b}{E_b} + \frac{\left(\frac{\nu_m}{E_m} - \frac{\nu_b}{E_b}\right)(1-\nu_b)}{(1-\nu_b) + \frac{E_b h}{E_m t}(1-\nu_m)}\right]$$

$$\text{or,}\quad \sigma_v = \frac{\xi_{tb}}{\left[\dfrac{\nu_b}{E_b} + \dfrac{\left(\dfrac{\nu_m}{E_m} - \dfrac{\nu_b}{E_b}\right)(1-\nu_b)}{(1-\nu_b) + \dfrac{E_b h}{E_m t}(1-\nu_m)} \right]} \tag{2.11}$$

Load:

$$P = \sigma_v B^2 \tag{2.12}$$

Substituting for σ_v in equation (2.12), we get

$$P = \frac{\xi_{tb} B^2}{\left[\dfrac{\nu_b}{E_b} + \dfrac{\left(\dfrac{\nu_m}{E_m} - \dfrac{\nu_b}{E_b}\right)(1-\nu_b)}{(1-\nu_b) + \dfrac{E_b h}{E_m t}(1-\nu_m)} \right]} \tag{2.13}$$

At failure, strain in brick reaches its ultimate value, ξ_u. So ultimate load at failure:

$$P_u = \frac{\xi_u B^2}{\left[\dfrac{\nu_b}{E_b} + \dfrac{\left(\dfrac{\nu_m}{E_m} - \dfrac{\nu_b}{E_b}\right)(1-\nu_b)}{(1-\nu_b) + \dfrac{E_b h}{E_m t}(1-\nu_m)} \right]} \tag{2.14}$$

The eqnation (2.14) has been used in calculating theoretical load capacities of columns. The average load capacity of axially loaded unreinforced columns is given in Table 4.14. The model that is presented here has to be drastically improved to compute the realistic collapse loads of brick columns. However, the model that is presented here is the first step in that direction.

CHAPTER III

EXPERIMENTAL PROGRAMME

3.1 TESTS ON MATERIALS

3.1.1 Mortar

Mortar cubes of size 75 mm were cast and tested after seven and 28 days. Standard sand has been used in mortar.

3.1.2 Brick

First class well burnt bricks were used in the brick masonry core construction. To study the dimensional characteristics, ten sets of values (each set for length, breadth and depth) were taken and the mean of these values is given in Table 4.1.

Compression tests on bricks were performed as per ISP:3495 (Part I) - 1976. Five samples of brick were soaked in water for one day before filling the frog with 1:2 mortar grout. The bricks were again soaked in water for three days before testing. Bricks were placed flat wise between two 3 mm plywood sheets and tested for the compressive strength.

Water absorption test of bricks was performed as specified by IS:3495 (Part II) - 1976. Five bricks were weighed and then immersed in water for 24 hours and then weighed again. The difference in weights indicated the amount of water absorbed.

TABLE 3.1

Details of Specimen Columns.

Specimen Column No.	Length (m)	Section (cm x cm)	Average comp. strength of bricks (MPa)	28 days cube strength of mortar in core (MPa)	28 days cube strength of mortar in plaster (MPa)	Skin rei forcemen details
L1	2.75	27.5x27.5	23.20	2.54	17.25	Nil
L2	2.75	27.5x27.5	23.20	2.54	17.25	Nil
S1	2.00	27.5x27.5	23.20	2.54	17.25	Nil
S2	2.00	27.5x27.5	23.20	2.54	17.25	Nil
S3	2.00	28 x 28	23.20	2.54	17.25	Two layer
S4	2.00	27.5x27.5	23.20	2.54	17.25	One layer
S5	2.00	25.2x25.2	23.20	2.54	17.25	Nil
S6	2.00	28 x 28	23.20	2.54	17.25	Two layer
S7	2.00	27.5x27.5	23.20	2.54	17.25	One layer
S8	2.00	25.2x25.2	23.20	2.54	17.25	Nil

Efflorescence test was performed as per IS:3495 (Part III) - 1976. A shallow dish was filled with distilled water. These five brick samples were placed in it in such a manner that the depth of immersion in water was more than 25 mm. The whole arrangement was kept until all the water in the dish was absorbed by the bricks and surplus water evaporated. A similar quantity of water was again poured in the dish and allowed to evaporate as before.

3.2 SPECIMEN PREPARATION

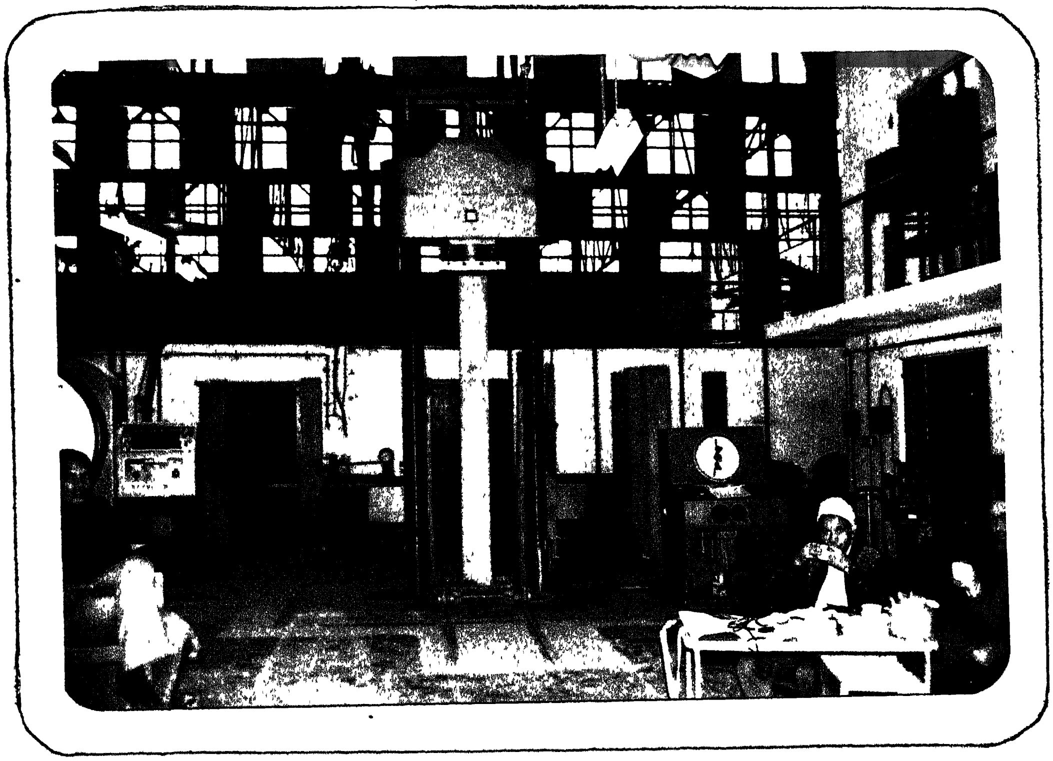
First, brick masonry cores were constructed. Then some of these were plastered without providing wire mesh reinforcement. Remaining cores were provided with ferrocement encasing by wrapping first, the wiremesh with the help of nails and binding wire and then plastering over it.

Columns S1, S2, L1, L2, S5 and S8 were not having wire mesh reinforcement. Columns were plastered with mortar prepared from cement of Batch-2. For core construction of all columns mortar of cement of Batch-1 was used.

Columns S4 and S7 were provided with two layers of wire mesh. The wrapping of wire mesh and plastering have been done after five days of completion of masonry cores, so that the cores could not be damaged while wrapping of mesh or plastering. In all columns 1:4 mortar has been used. The details of specimens are given in Table 3.1.

3.3 TESTING OF COLUMNS

The experiments were planned to determine the bearing capacity and deformability of columns loaded by an axial or eccentric force of short duration. Axial loading has been given as UDL by putting a plate over column and then loading on it. Eccentric loading has been provided by plate and rollar arrangement. A rubber sheet was also put in each case between column surface and plate. Loading arrangements are shown in Figs. 3.1 through 3.3 by photographs.



ig. 3.1: Photograph showing test set-up for axial loading in compression testing machine.

g. 3.2: Photograph showing test set-up for axial loading in Tinius Olsen Universal Testing Machine.

Fig. 3.3: Photograph showing test set-up for eccentric loading in Tinius Olsen Universal Testing Machine.

Possibly large programme of observations of the physical phenomenon which accompany the loading and the failure of the columns has been realised. The load was applied at the rate of 9810 kN per minute. While testing, it was observed that most of the columns having no reinforcement failed locally from the top. Some of these columns have been tested at different stages after removing the top portion which broke down and making the top almost plane. These shortened structures of the specimen columns have been called columns of the series of their original specimen columns. The columns tested in more than one stage are L1, L2, S1 and S2. Various series and their columns are as follows:

Name of the Series	Columns of the Series
L1	L1a, L1b, L1c, L1d
L2	L2a, L2b, L2c, L2d
S1	S1a, S1b
S2	S2a, S2b

The two columns L1 and L2 have been tested in Compression Testing Machine having capacity 600 tonnes and all other columns were tested in Tinius Olsen Universal Testing Machine having capacity 200 tonnes. Each column has been tested on the fortieth day of its completion.

For observing strains a gauge length of 28 cm was used in each case.

CHAPTER IV

RESULTS AND DISCUSSION

4.1 TESTS ON MATERIALS

4.1.1 Mortar

Batch - 1

(i) 7 - days cube strength = 1.4 MPa

(ii) 28 - days cube strength = 2.54 MPa

Batch - 2

(i) 7 - days cube strength = 11.47 MPa

(ii)28 - days cube strength = 17.25 MPa

4.1.2 Bricks

(a) Dimensional Characteristics of Bricks

Table 4.1 shows the dimensional characteristics of bricks. It is observed that as dimensions increase, coefficients of variation decrease. Secondly the coefficient of variation is in the range of 1% to 5%, which is very good for hand made units.

TABLE 4.1

Dimensional Characteristics of Bricks.

Dimension	Mean (mm)	Standard deviation	Coefficient of variation
Length	228.00	2.65	1.16
Breadth	107.50	3.50	3.25
Depth	62.10	2.98	4.80

(b) <u>Compressive Strength</u>

The mean compressive strength of bricks was found to be 23.20 MPa with a standard deviation, 1.50.

(c) <u>Water Absorption</u>

The average of water absorption was found to be 14.1% by weight which is with in the limits specified in IS:1077-1976.

(d) <u>Efflorescence</u>

There was no perceptible deposit of efflorescence. Therefore, the rating of efflorescence was reported as 'Nil' in accordance with the definition stated in IS:3495 (Part III) - 1976.

4.2 TESTS ON COLUMNS

Observations taken during the experiments have been presented in Table Nos. 4.2 through 4.15. For axially loaded columns load versus axial deformation and axial deformation versus lateral dimension, graphs at different loads, are plotted. For eccentrically loaded columns graphs of axial deformation versus

lateral dimension are plotted.

All these graphs are shown in Figs. 4.16 through 4.30. Failure modes of different columns are shown in Figs. 4.1 through 4.15 with the help of photographs.

In all the columns failure started with vertical splitting of bricks. Thirteen columns were tested under axial loading including eleven unreinforced and two single layered reinforced columns. All of the unreinforced columns failed locally from top. One of two reinforced columns in which only 2 cm horizontal overlap was given betwen two meshes, failed by buckling from middle followed by splitting and crushing of bricks from top and bottom. Remaining one reinforced column having 15 cm horizontal overlap between meshes started failing from top as well as bottom simultaneously and finally failed only at the ends.

Five columns were tested under eccentric loading, out of which three columns were without reinforcement and two were with two layers of mesh reinforcement. One of the unreinforced column failed locally from bottom and remaining two of them failed from top.

It seems that stress concentration occurs near the supports and most of the columns have failed from top due to nonuniform loading because the top of the column might not be having perfect plane surface leading to improper contact with loading platten.

Columns L1 and L2 failed locally from top upto not more than 20% of their lengths. But when shortened columns were tested, the

TABLE No.4.2

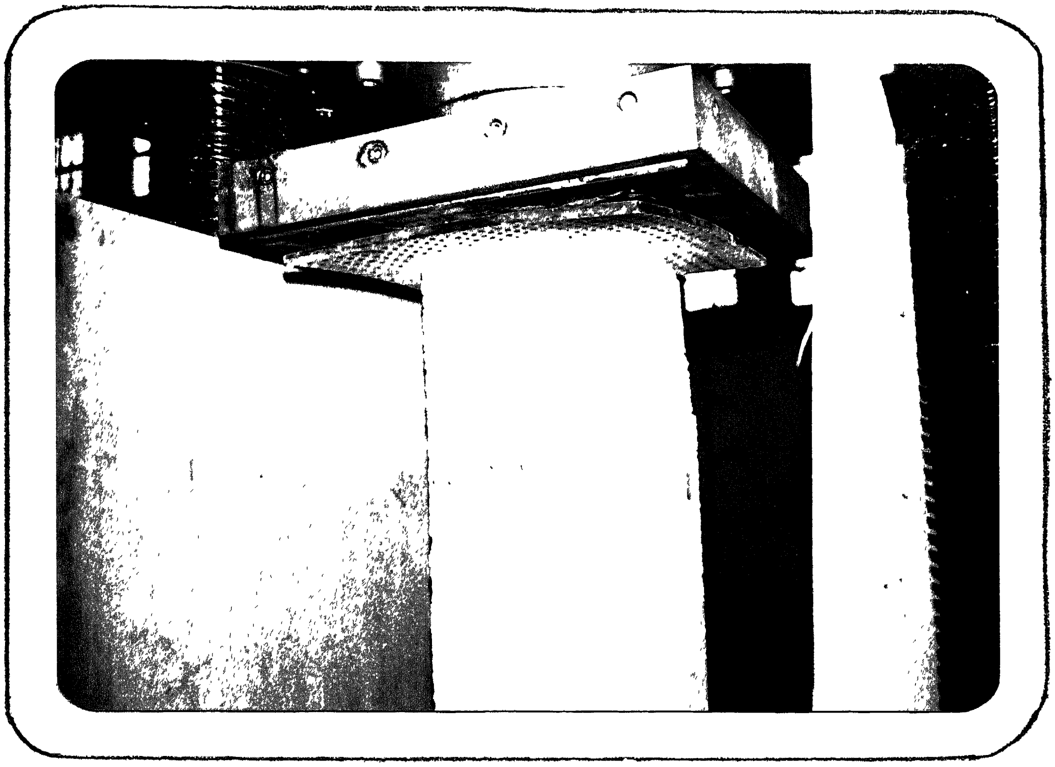
RESULTS OF AXIALLY LOADED COLUMNS OF SERIES L1

COLUMN No.	LENGTH in METRES	LOAD in kN	PHOTOGRAPH No.	REMARKS
L1a	2.75	122.62 158.92	4.1, 4.2	First Crack Failure
L1b	2.11	147.15 206.60	4.3	First Crack Failure
L1c	1.72	188.35 194.24	4.4	First crack Failure
L1d	1.23	294.30 250.15 161.8	4.5 4.6	First crack Crack widened Failure

Ph.No. 4.1

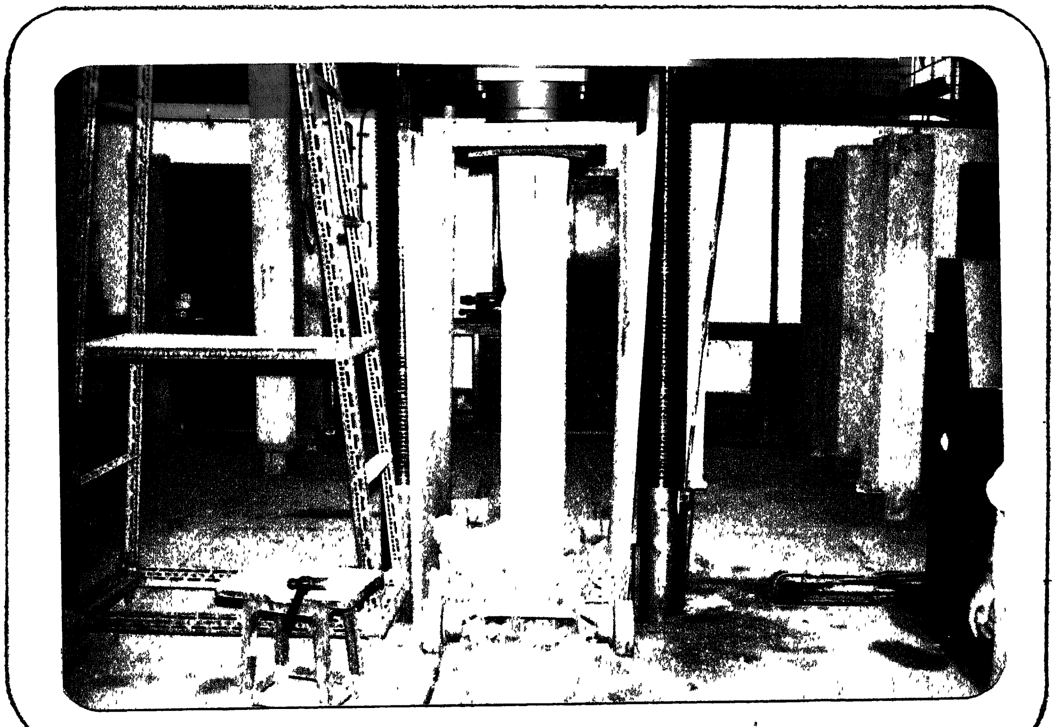
Ph.No. 4.2

Fig. 4.1: Failure mode of axially loaded column L1a



Ph.No. 4.3

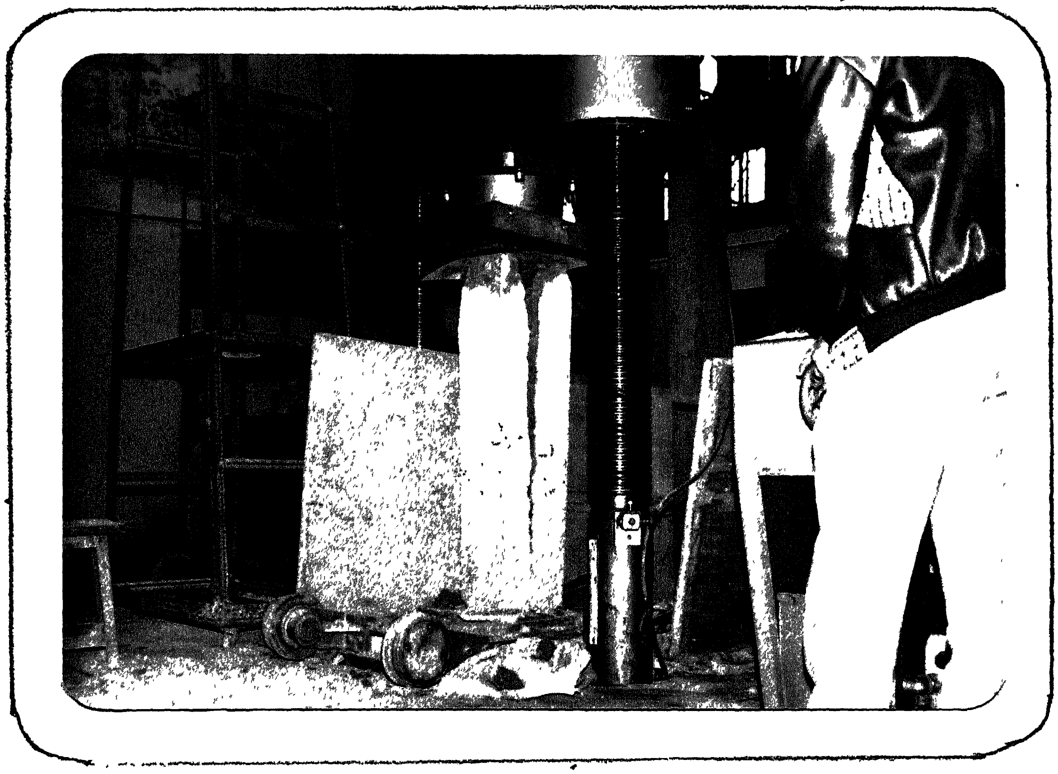
Fig. 4.2: Failure mode of axially loaded column L1b



Ph.No. 4.4

Fig. 4.3: Failure mode of axially loaded column L1c.

Ph.No. 4.5



Ph.No. 4.6

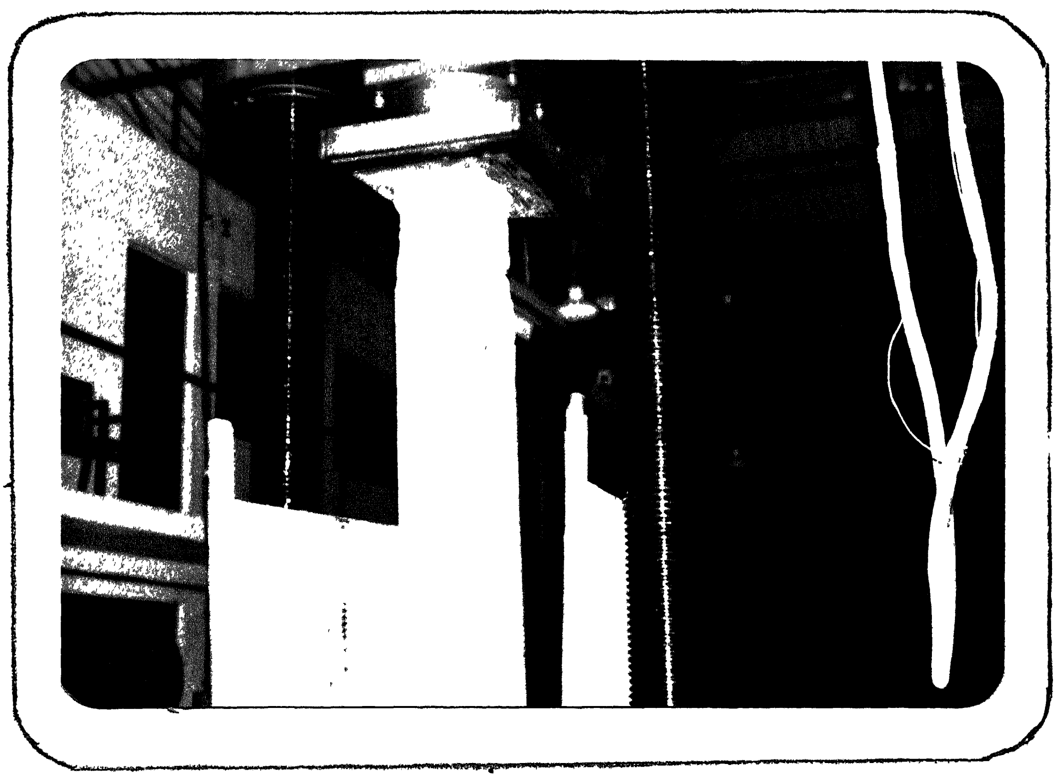
Fig. 4.4: Failure mode of axially loaded column L1d

TABLE No. 4.3

RESULTS OF AXIALLY LOADED COLUMNS OF SERIES L2

COLUMN No.	LENGTH in METRES	LOAD in kN	PHOTOGRAPH No.	REMARKS
L2a	2.75	132.43 176.58	4.7 , 4.8	First Crack Failure
L2b	2.13	156.96 210.92		First Crack Failure
L2c	1.70	221.71 196.79		First crack Failure
L2d	1.21	284.49 304.11		First Crack Failure

Ph.No. 4.7



Ph.No. 4.8

Fig. 4.5: Failure mode of axially loaded column L2a

TABLE No. 4.4

COLUMN No.S2a

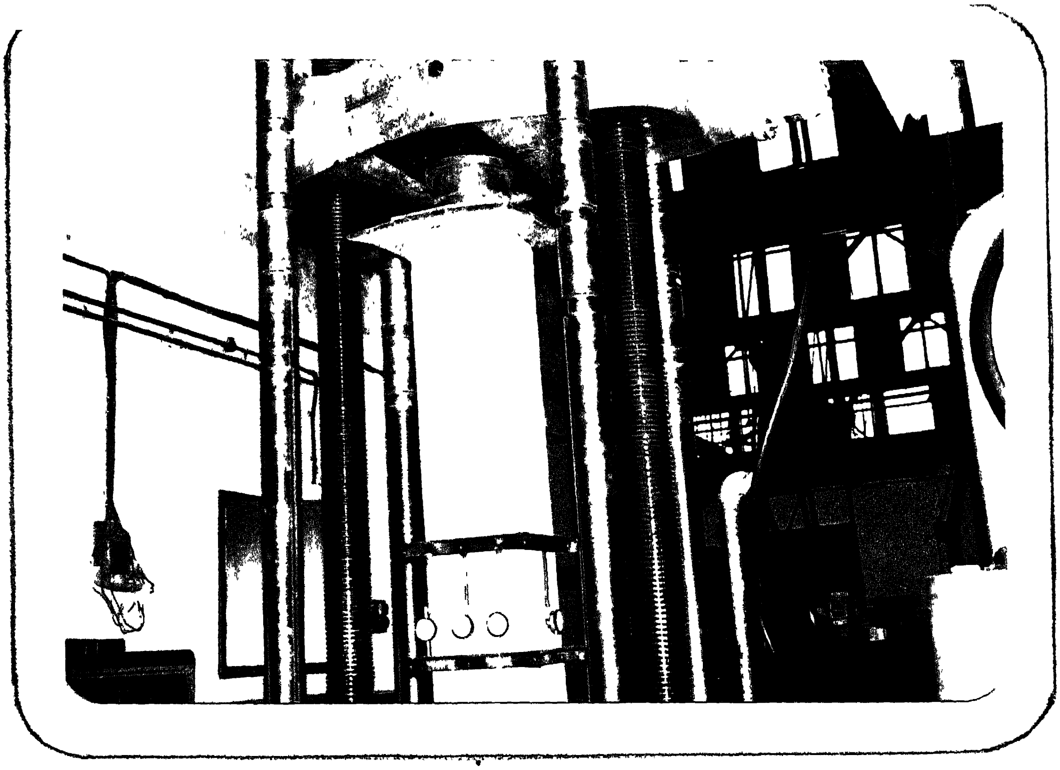
B/L = 0.137

TYPE OF LOADING : AXIAL

LOAD IN kN	ΔX_1 (mm)	ΔX_2 (mm)	ΔX_3 (mm)	ΔX_4 (mm)	ΔX_5 (mm)	ΔX_6 (mm)	Av. ΔX (mm)	Ph. No.	REMARKS
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		
73.57	0.07	0.07	0.06	0.05	0.03	0.06	0.06		
98.10	0.10	0.09	0.08	0.07	0.06	0.10	0.08		
122.6	0.13	0.12	0.11	0.09	0.07	0.12	0.11		
147.1	0.17	0.17	0.15	0.11	0.10	0.16	0.14		First Crack
166.7	0.19	0.18	0.16	0.13	0.10	0.17	0.16	4.9	
147.2	0.20	0.19	0.17	0.13	0.12	0.18	0.17	4.10	Failure

ΔX_1 , ΔX_2 , ΔX_3 , ΔX_4 , ΔX_5 and ΔX_6 are Axial Deformations.

Ph : Photograph



Ph.No. 4.9

Ph.No. 4.10

Fig. 4.6: Failure mode of axially loaded column S2a

TABLE No. 4.5

COLUMN No. S2b

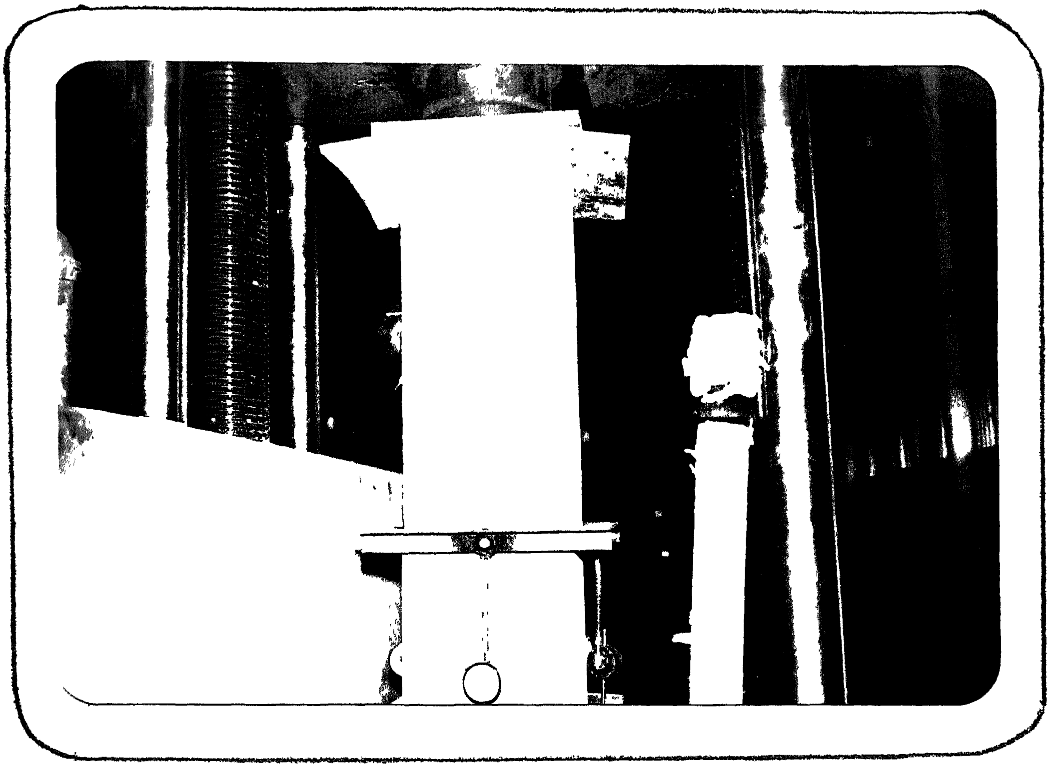
B/L = 0.189

TYPE OF LOADING : AXIAL

LOAD IN kN	ΔX_1 (mm)	ΔX_2 (mm)	ΔX_3 (mm)	ΔX_4 (mm)	ΔX_5 (mm)	ΔX_6 (mm)	Av. ΔX (mm)	Ph. No.	REMARKS
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		
24.52	0.02	0.03	0.03	0.03	0.02	0.02	.025		
49.05	0.04	0.05	0.05	0.05	0.03	0.03	0.04		
73.57	0.05	0.07	0.07	0.08	0.05	0.04	0.06		
98.10	0.07	0.09	0.09	0.10	0.07	0.06	0.08		
122.6	0.08	0.11	0.11	0.13	0.09	0.07	0.10		
147.2	0.10	0.12	0.13	0.15	0.12	0.08	0.12		
171.6	0.12	0.16	0.16	0.18	0.14	0.11	0.14	4.11	First Crack
165.7									
186.4								4.12	Failure

ΔX_1 , ΔX_2 , ΔX_3 , ΔX_4 , ΔX_5 and ΔX_6 are Axial Deformations.

Ph : Photograph



Ph.No. 4.11

Ph.No. 4.12

Fig. 4.7: Failure mode of axially loaded column S2b

TABLE No. 4.6

COLUMN No. S5

B/L = 0.126

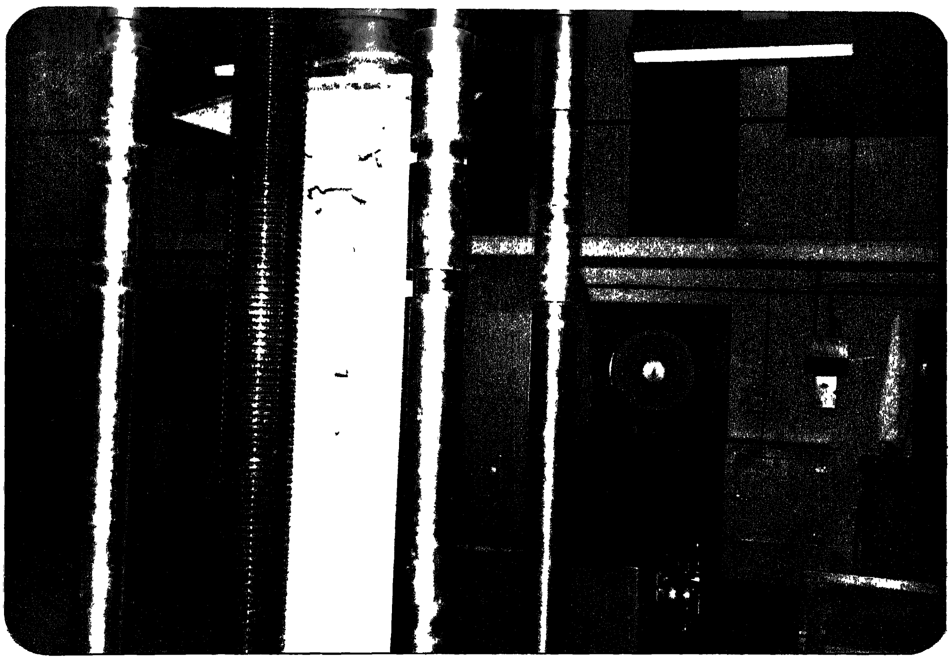
TYPE OF LOADING : AXIAL

LOAD IN kN	ΔX_1 (mm)	ΔX_2 (mm)	ΔX_3 (mm)	ΔX_4 (mm)	ΔX_5 (mm)	ΔX_6 (mm)	Av. ΔX (mm)	Ph. No.	REMARKS
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		
24.52	0.01	0.02	0.02	0.02	0.02	0.01	0.02		
49.05	0.02	0.04	0.04	0.04	0.03	0.02	0.03		
73.57	0.04	0.06	0.07	0.07	0.05	0.04	0.05		
98.10	0.07	0.09	0.10	0.09	0.06	0.06	0.08		
122.6	0.09	0.11	0.12	0.12	0.08	0.07	0.09		
147.2	0.11	0.14	0.15	0.15	0.11	0.09	0.13		First crack
171.6	0.10	0.16	0.20	0.20	0.14	0.07	0.14		
196.2	0.12	0.14	0.21	0.24	0.21	0.09	0.17	4.13	
220.7	0.14	0.12	0.21	0.25	0.25	0.10	0.18	4.14	Failure

ΔX_1 , ΔX_2 , ΔX_3 , ΔX_4 , ΔX_5 and ΔX_6 are Axial Deformations.

Ph : Photograph

Ph.No. 4.13



Ph.No. 4.14

Fig. 4.8: Failure mode of axially loaded column S5

TABLE No. 4.7

COLUMN No. S4

B/L = 0.137

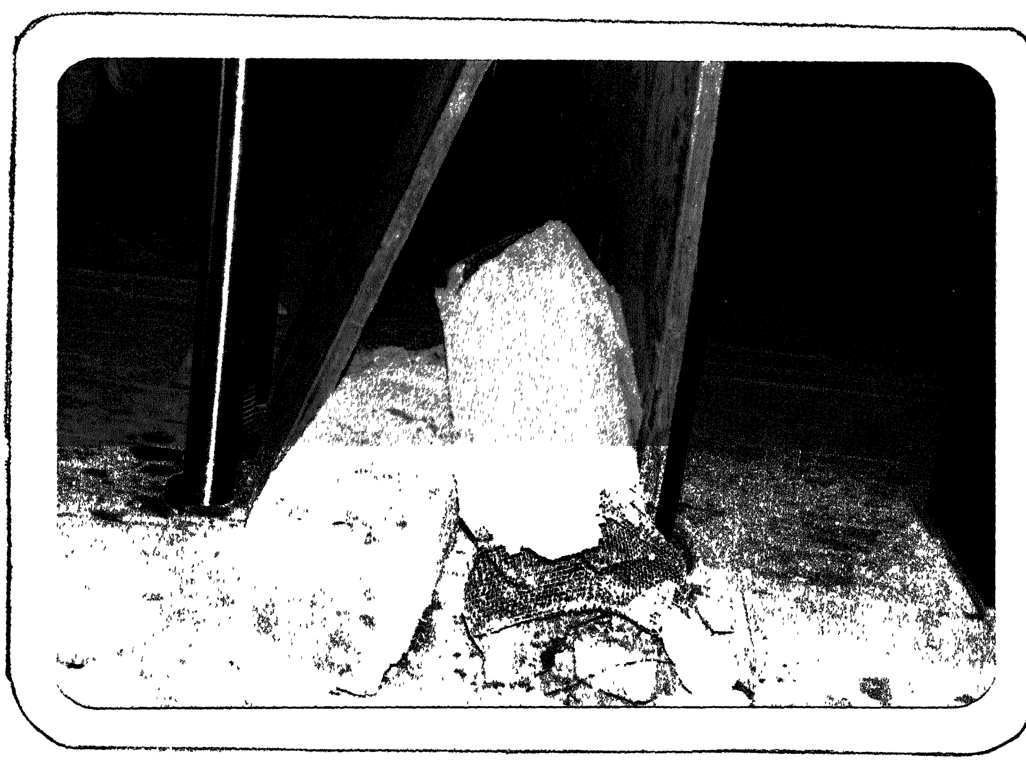
TYPE OF LOADING : AXIAL

LOAD IN kN	ΔX_1 (mm)	ΔX_2 (mm)	ΔX_3 (mm)	ΔX_4 (mm)	ΔX_5 (mm)	ΔX_6 (mm)	Av. ΔX (mm)	Ph. No.	REMARKS
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		
24.52	0.01	0.02	0.01	0.01	0.01	0.01	0.01		
49.05	0.03	0.03	0.03	0.03	0.03	0.01	.026		
73.57	0.05	0.05	0.04	0.05	0.05	0.03	0.04		
98.10	0.07	0.07	0.06	0.06	0.07	0.05	0.06		
122.6	0.09	0.10	0.08	0.08	0.09	0.08	0.08		
147.1	0.12	0.12	0.10	0.10	0.11	0.11	0.11		
171.7	0.15	0.15	0.14	0.13	0.14	0.14	0.14		
187.8	0.30	0.26	0.17	0.13	0.16	0.25	0.21		First Crack
196.2	0.40	0.37	0.25	0.21	0.16	0.30	0.28		
208.5	0.56	0.55	0.41	0.32	0.16	0.36	0.39		
220.7	0.68	0.67	0.52	0.43	0.17	0.42	0.48	4.15	Failure
								4.16	

ΔX_1 , ΔX_2 , ΔX_3 , ΔX_4 , ΔX_5 and ΔX_6 are Axial Deformations.

Ph : Photograph

Ph.No. 4.15



Ph.No. 4.16

Fig. 4.9: Failure mode of axially loaded column S4

TABLE No. 4.8

COLUMN No. S7

B/L = 0.137

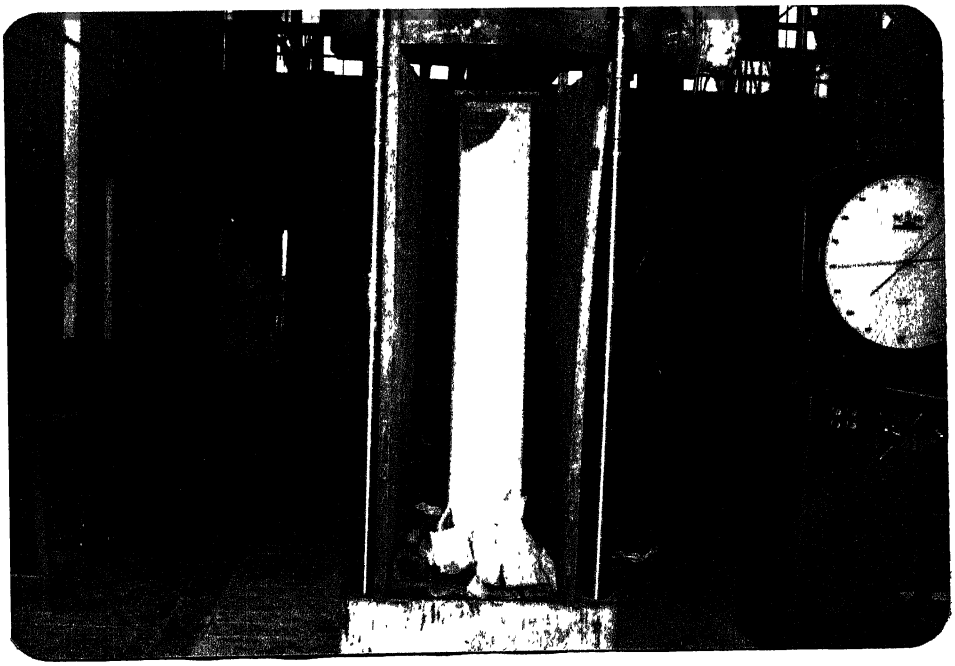
TYPE OF LOADING : AXIAL

LOAD IN kN	ΔX_1 (mm)	ΔX_2 (mm)	ΔX_3 (mm)	ΔX_4 (mm)	ΔX_5 (mm)	ΔX_6 (mm)	Av. ΔX (mm)	Ph. No.	REMARKS
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		
24.52	0.00	0.01	0.02	0.02	0.02	0.01	0.01		
49.05	0.02	0.03	0.04	0.05	0.05	0.02	0.03		
73.57	0.04	0.05	0.06	0.07	0.06	0.05	0.05		
98.10	0.07	0.08	0.09	0.09	0.07	0.06	0.07		
122.6	0.09	0.11	0.11	0.10	0.09	0.08	0.09		
147.2	0.09	0.12	0.13	0.11	0.10	0.10	0.11		
171.7	0.08	0.12	0.14	0.11	0.12	0.13	0.12		
196.2	0.06	0.11	0.13	0.10	0.13	0.14	0.11		First Crack
208.5	0.03	0.07	0.11	0.08	0.14	0.13	0.10	4.17	Failure
								4.18	

ΔX_1 , ΔX_2 , ΔX_3 , ΔX_4 , ΔX_5 and ΔX_6 are Axial Deformations.

Ph : Photograph

Ph.No. 4.17



Ph.No. 4.18

Fig. 4.10: Failure mode of axially loaded column

TABLE No. 4.9

COLUMN No. S1a

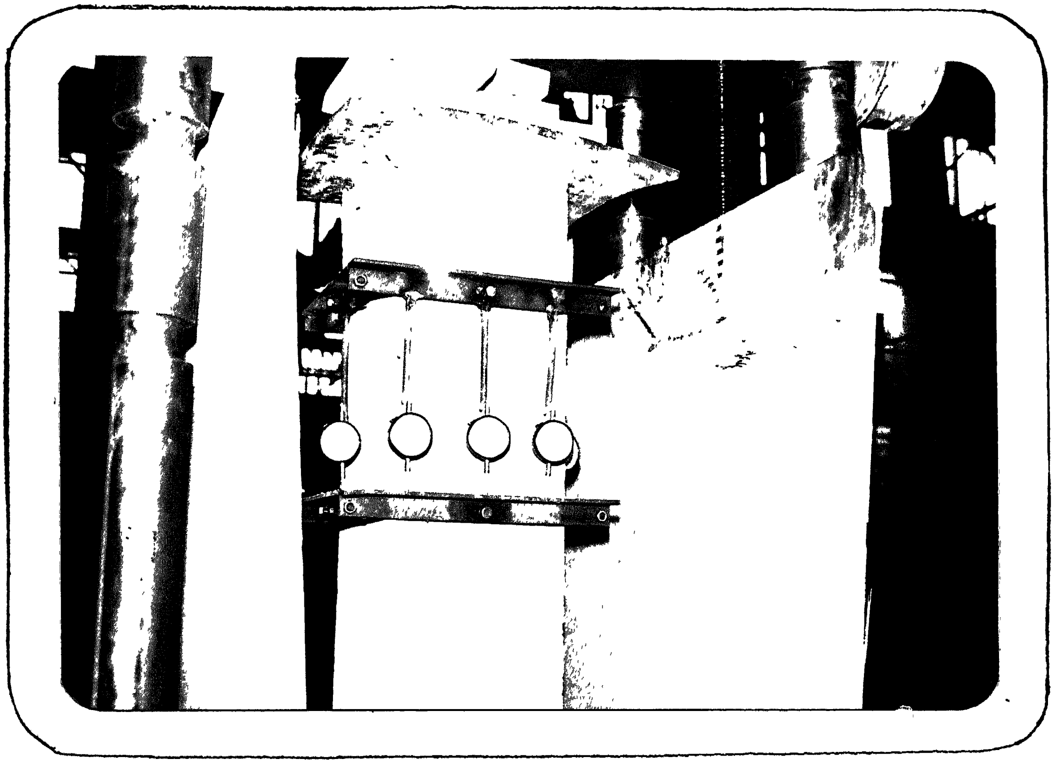
B/L = 0.137

TYPE OF LOADING : ECCENTRIC (e= 4.5 cm.)

LOAD IN kN	ΔX_1 (mm)	ΔX_2 (mm)	ΔX_3 (mm)	ΔX_4 (mm)	ΔX_5 (mm)	ΔX_6 (mm)	Ph. No.	REMARKS
0.00	0.00	0.00	0.00	0.00	0.00	0.00		
24.52	0.03	0.00	0.00	0.01	0.01	0.03		
49.05	0.06	0.03	0.01	0.01	0.03	0.07		
73.57	0.09	0.06	0.03	0.02	0.05	0.11		
98.10	0.13	0.08	0.04	0.02	0.08	0.16		
115.3	0.16	0.10	0.15	0.03	0.12	0.20		
122.6	0.27	0.18	0.09	0.03	0.18	0.31		First Crack
144.7							4.19	Failure

ΔX_1 , ΔX_2 , ΔX_3 , ΔX_4 , ΔX_5 and ΔX_6 are Axial Deformations.

Ph : Photograph



Ph.No. 4.19

Fig. 4.11: Failure mode of eccentrically loaded column S1.

TABLE No. 4.10

COLUMN No. S1b

B/L= 0.155

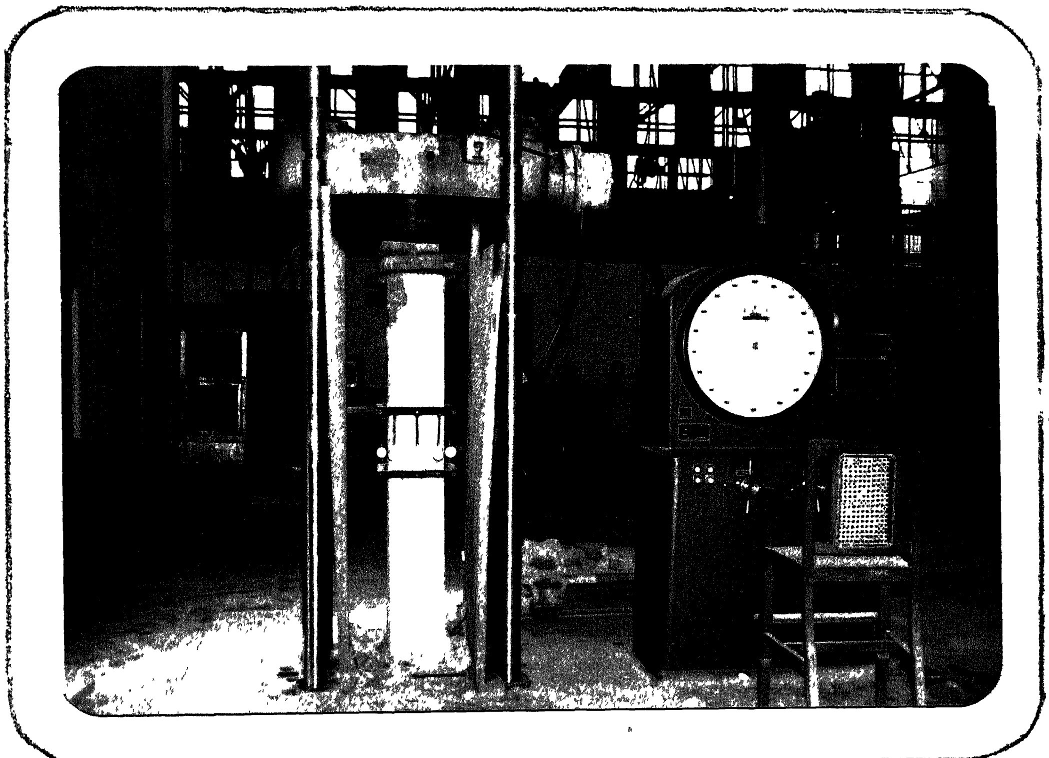
TYPE OF LOADING : ECCENTRIC (e= 2.5 cm.)

LOAD IN kN	ΔX_1 (mm)	ΔX_2 (mm)	ΔX_3 (mm)	ΔX_4 (mm)	ΔX_5 (mm)	ΔX_6 (mm)	Ph. No.	REMARKS
0.00	0.00	0.00	0.00	0.00	0.00	0.00		
24.52	0.03	0.03	0.03	0.02	0.01	0.04		
49.05	0.07	0.05	0.05	0.03	0.03	0.07		
73.57	0.10	0.09	0.08	0.05	0.05	0.10		
98.10	0.13	0.11	0.10	0.06	0.07	0.13		
122.6	0.15	0.13	0.11	0.08	0.08	0.16		
147.2	0.18	0.15	0.13	0.09	0.10	0.18		
171.6	0.22	0.19	0.17	0.12	0.12	0.22		First Crack
199.1	0.30	0.36	0.26	0.16	0.14	0.24	4.20, 4.21	Failure

ΔX_1, ΔX_2, ΔX_3, ΔX_4, ΔX_5 and ΔX_6 are Axial Deformations.

Ph : Photograph

Ph.No. 4.20



Ph.No. 4.21

Fig. 4.12: Failure mode of eccentrically loaded column S1b

TABLE No. 4.11

COLUMN No. S8

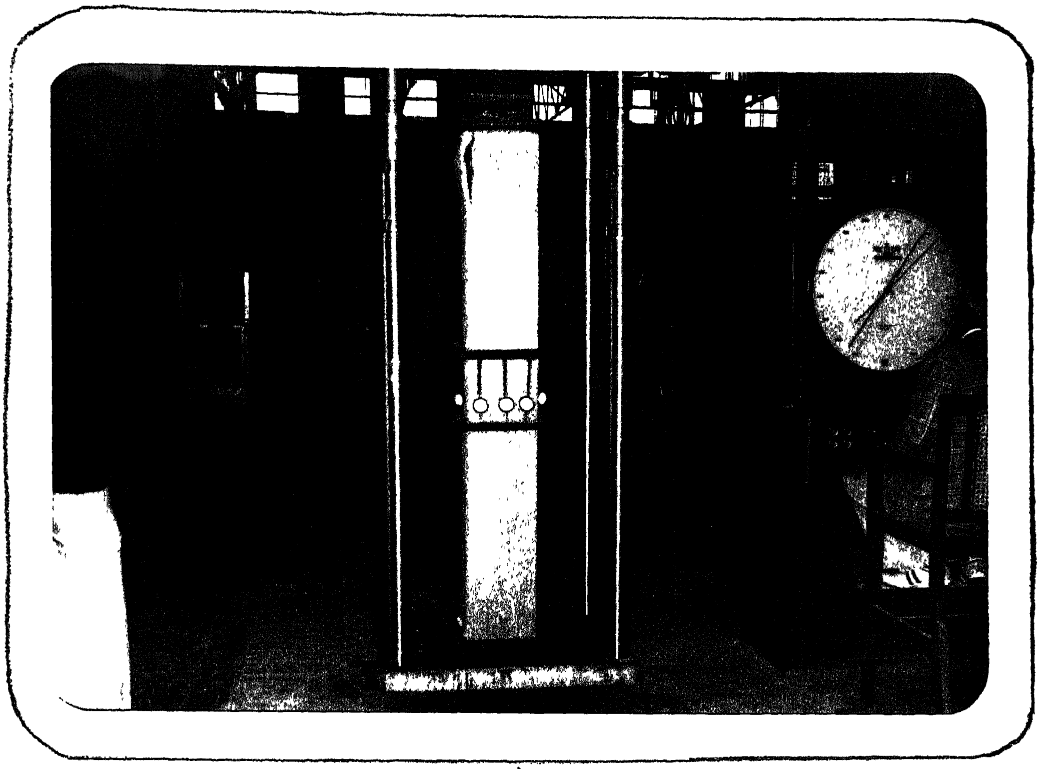
B/L= 0.126

TYPE OF LOADING : ECCENTRIC (e= 1.2 cm.)

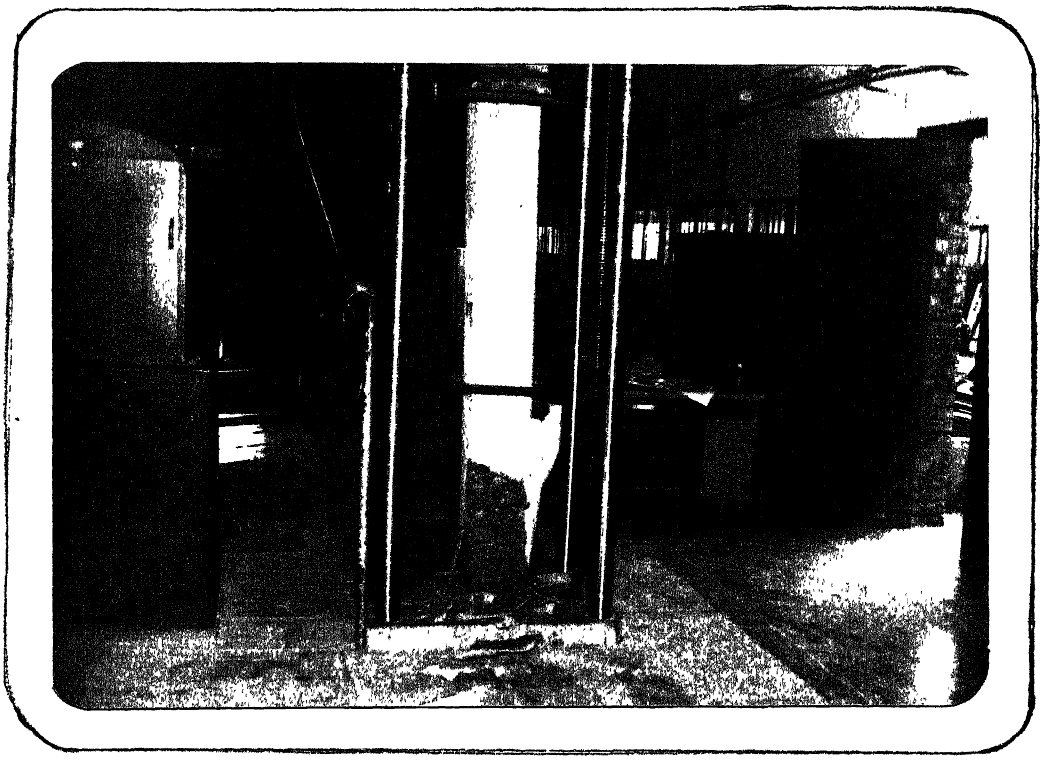
LOAD IN kN	ΔX_1 (mm)	ΔX_2 (mm)	ΔX_3 (mm)	ΔX_4 (mm)	ΔX_5 (mm)	ΔX_6 (mm)	Ph. No.	REMARKS
0.00	0.00	0.00	0.00	0.00	0.00	0.00		
24.52	0.01	0.02	0.02	0.03	0.02	0.01		
49.05	0.01	0.04	0.04	0.06	0.03	0.02		
73.57	0.02	0.06	0.07	0.09	0.07	0.03		
98.10	0.04	0.08	0.10	0.13	0.09	0.05		
122.6	0.09	0.13	0.14	0.15	0.10	0.07		First Crack
147.2	0.14	0.18	0.19	0.18	0.10	0.12		
171.6	0.22	0.24	0.24	0.21	0.12	0.17	4.22 . 4.23	Failure

ΔX_1 , ΔX_2 , ΔX_3 , ΔX_4 , ΔX_5 and ΔX_6 are Axial Deformations.

Ph : Photograph



Ph.No. 4.22



Ph.No. 4.23

Fig. 4.13: Failure mode of eccentrically loaded column S8

TABLE No.4.12

COLUMN No.S3

B/L = 0.14

TYPE OF LOADING : ECCENTRIC (e= 4.5 cm.)

LOAD IN kN	ΔX_1 (mm)	ΔX_2 (mm)	ΔX_3 (mm)	ΔX_4 (mm)	ΔX_5 (mm)	ΔX_6 (mm)	Ph. No.	REMARKS
0.00	0.00	0.00	0.00	0.00	0.00	0.00		
24.52	0.02	0.00	0.00	0.00	0.01	0.03		
49.05	0.05	0.02	0.00	0.01	0.03	0.05		
73.57	0.07	0.03	0.02	0.01	0.04	0.08		
98.10	0.10	0.05	0.03	0.01	0.04	0.12		
122.6	0.12	0.06	0.03	0.02	0.04	0.15		
147.2	0.15	0.09	0.05	0.06	0.06	0.18		
171.6	0.18	0.11	0.06	0.04	0.09	0.22		First Crack
196.2	0.22	0.14	0.09	0.05	0.11	0.26		
220.7	0.27	0.18	0.12	0.06	0.12	0.30	4.24	
242.8	0.29	0.20	0.14	0.08	0.14	0.33		
255.1							4.25	Failure

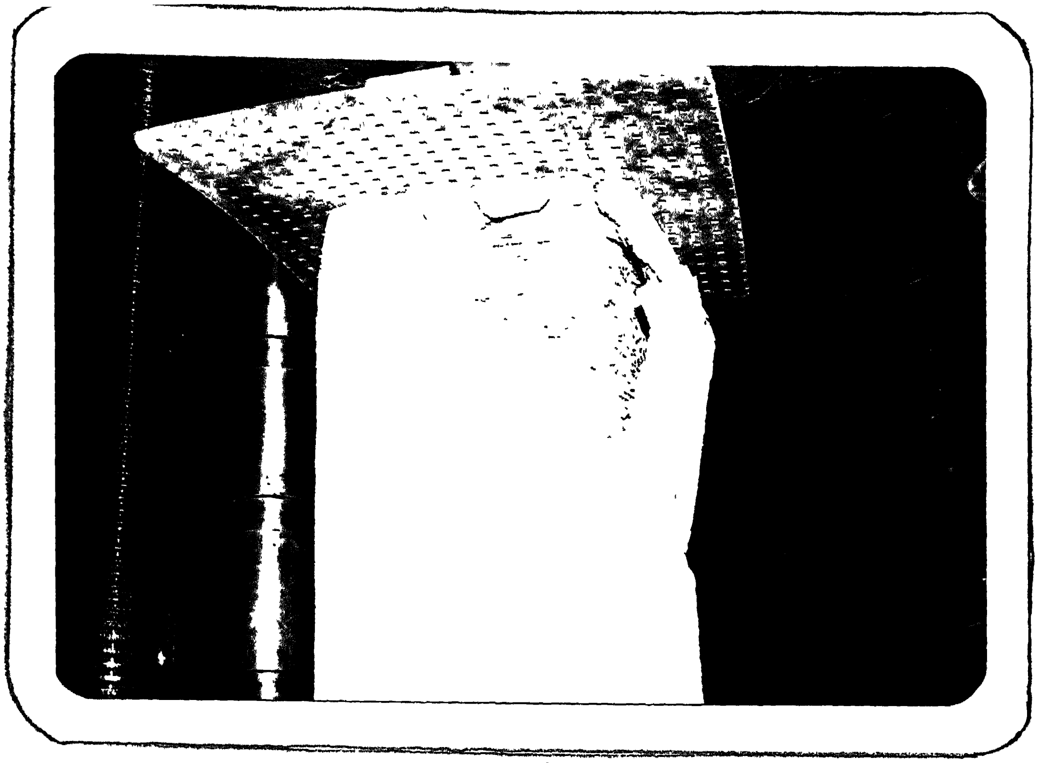
ΔX_1 , ΔX_2 , ΔX_3 , ΔX_4 , ΔX_5 and ΔX_6 are Axial Deformations.

Ph : Photograph

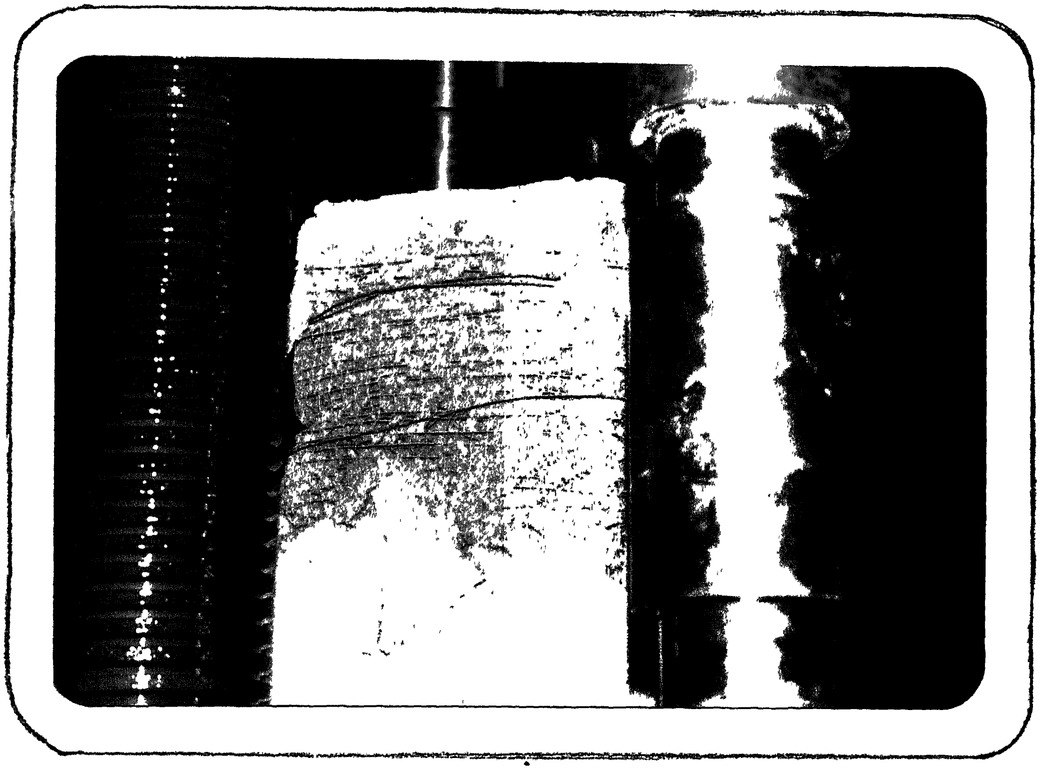
Ph.No. 4.24

Ph.No. 4.25

Fig. 4.14: Failure mode of eccentrically loaded column S3

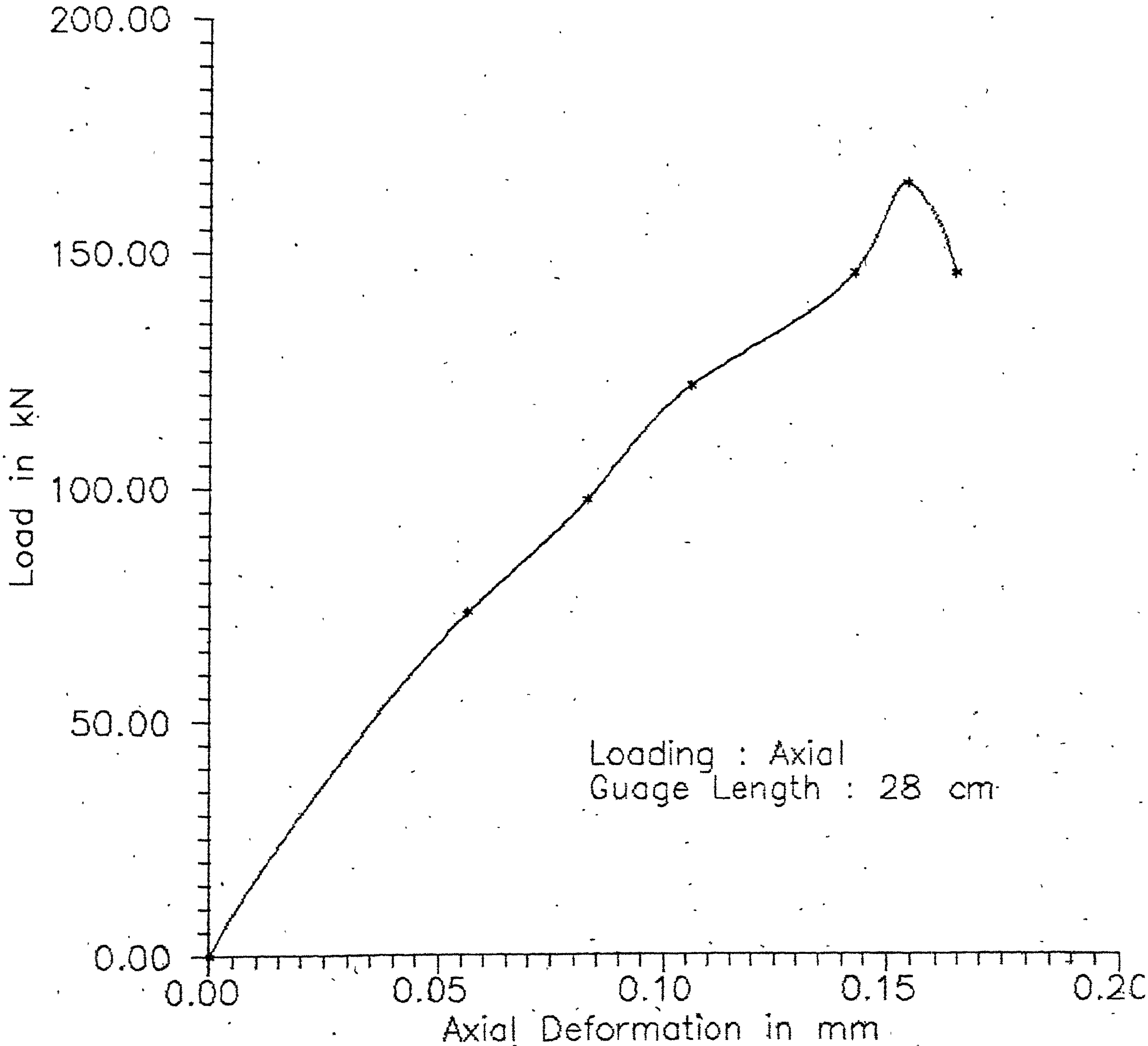


Ph.No. 4.26



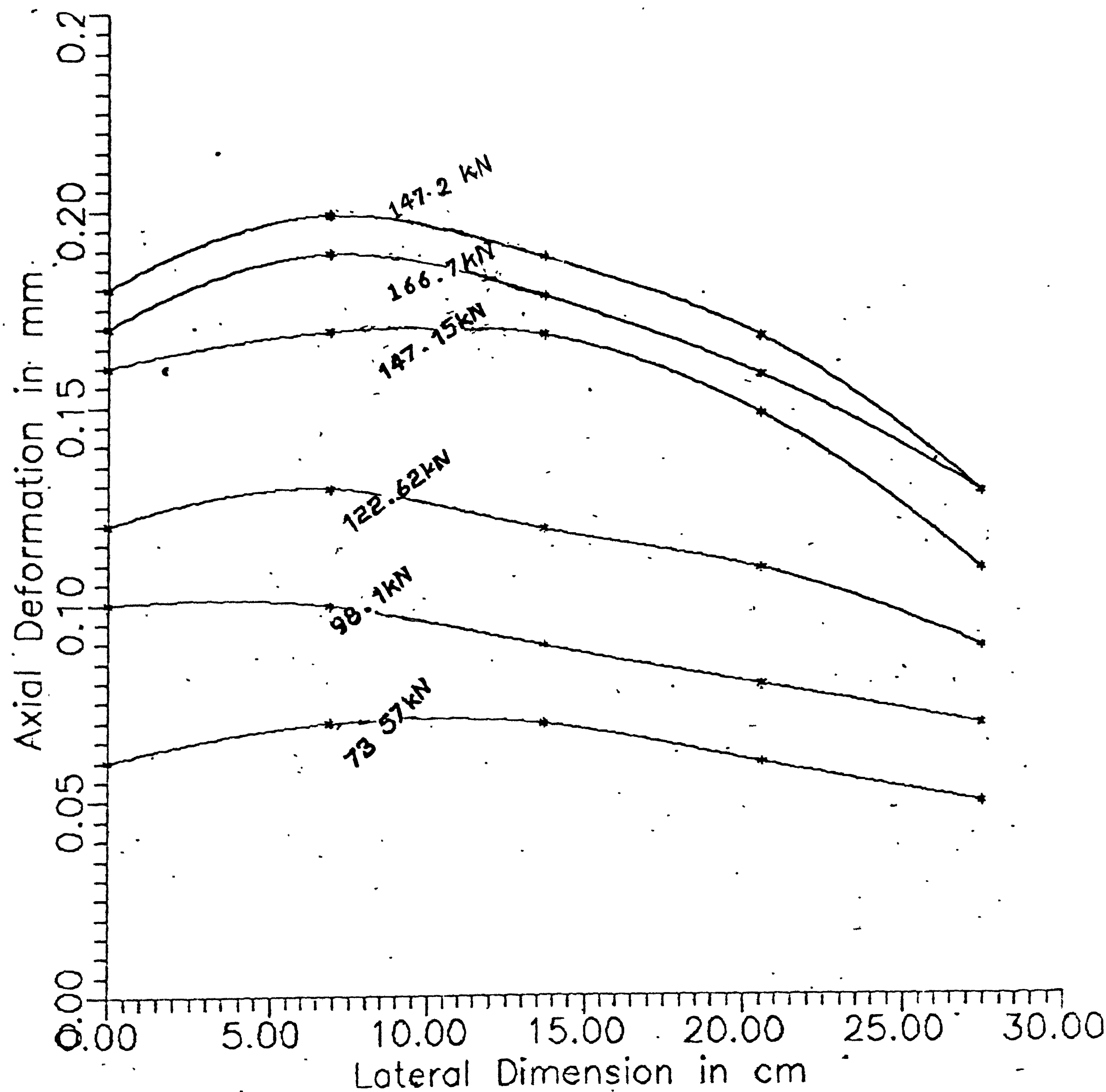
Ph.No. 4.27

Fig. 4.15: Failure mode of eccentrically loaded column S6



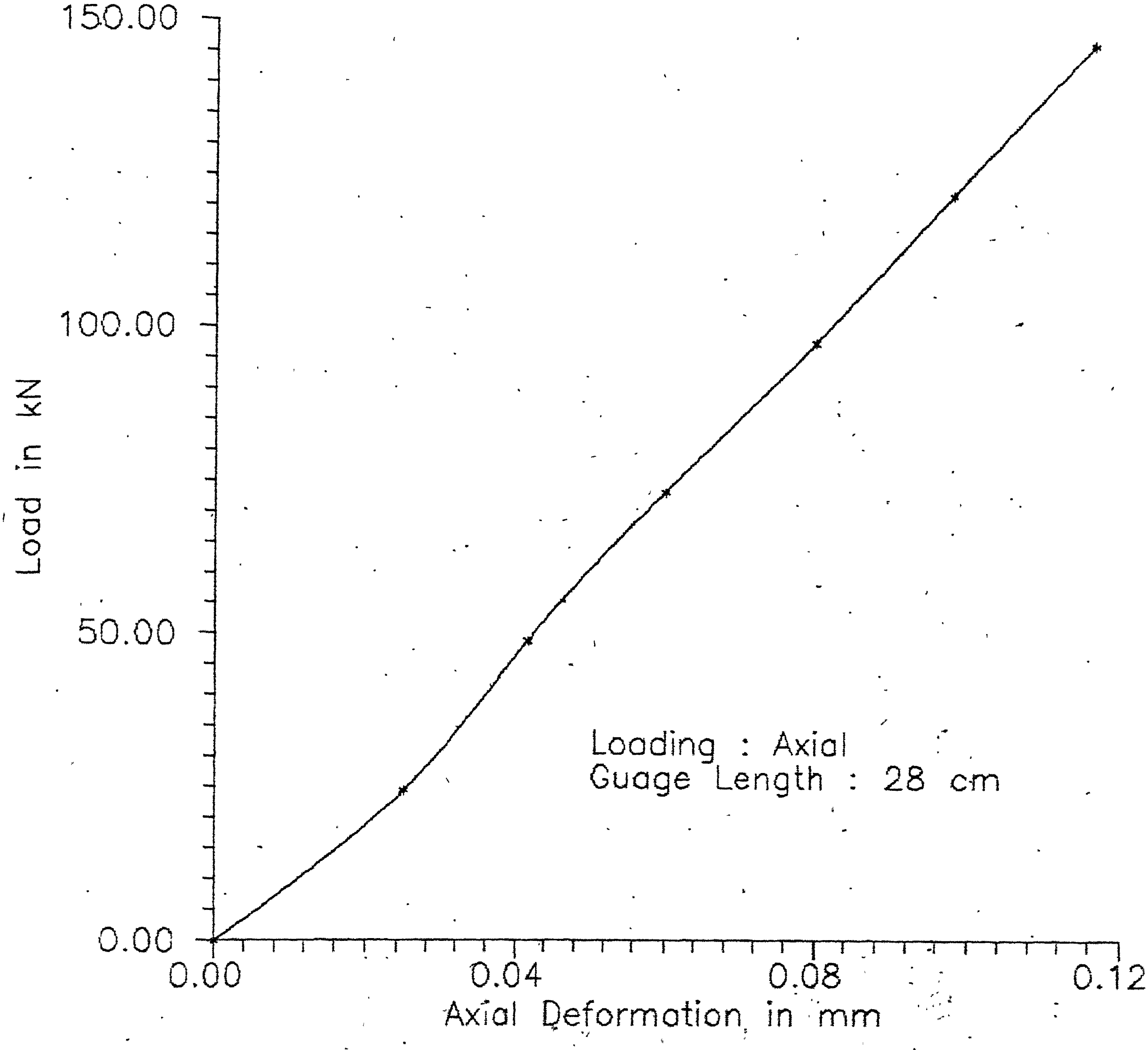
Load/Axial Deformation Curve,
COLUMN-S2a

FIG 4.16



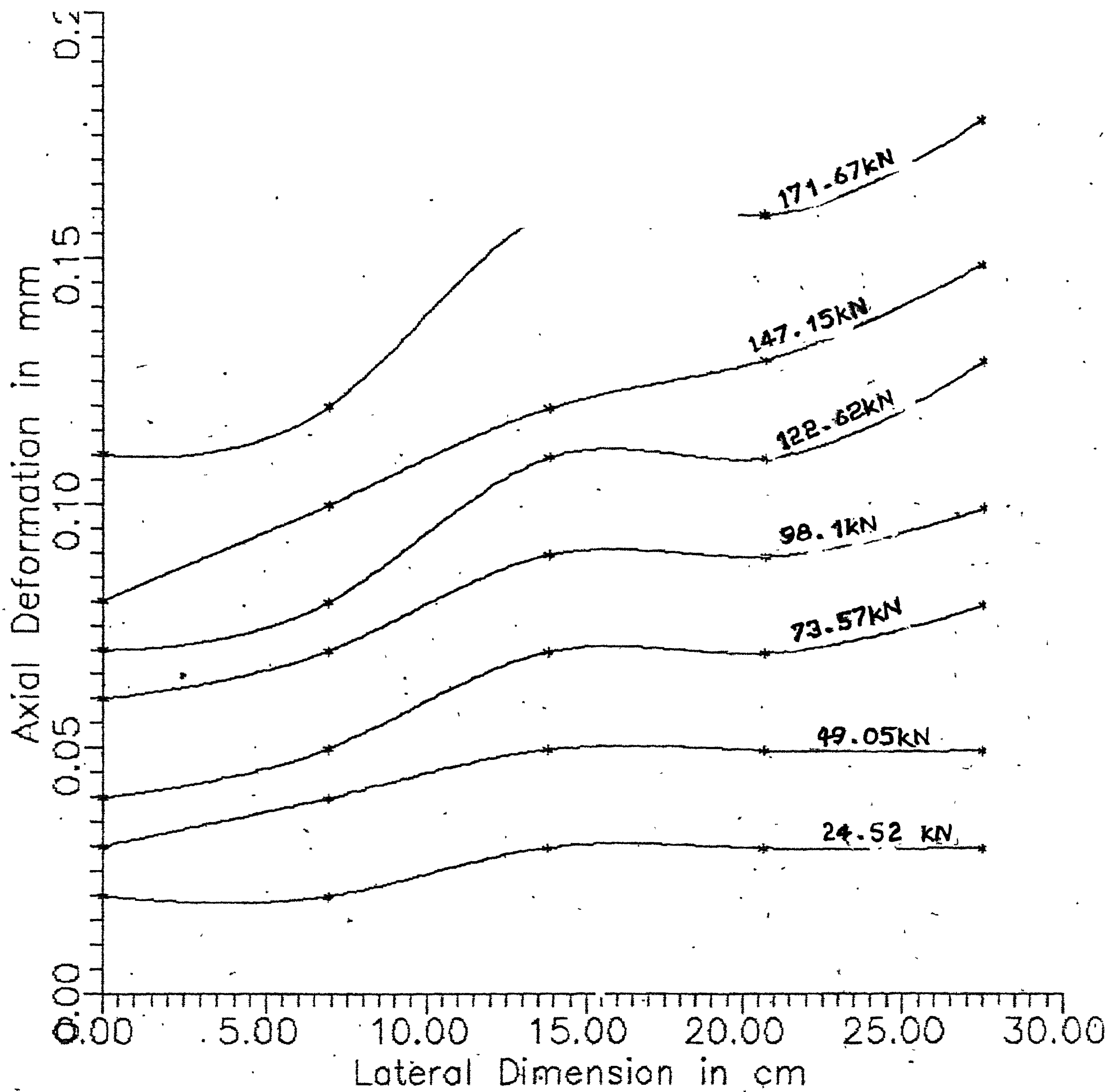
Axial Deformation/Lateral Dimension (at L.D.)
Curve, COLUMN-S2a

FIG. 4.17



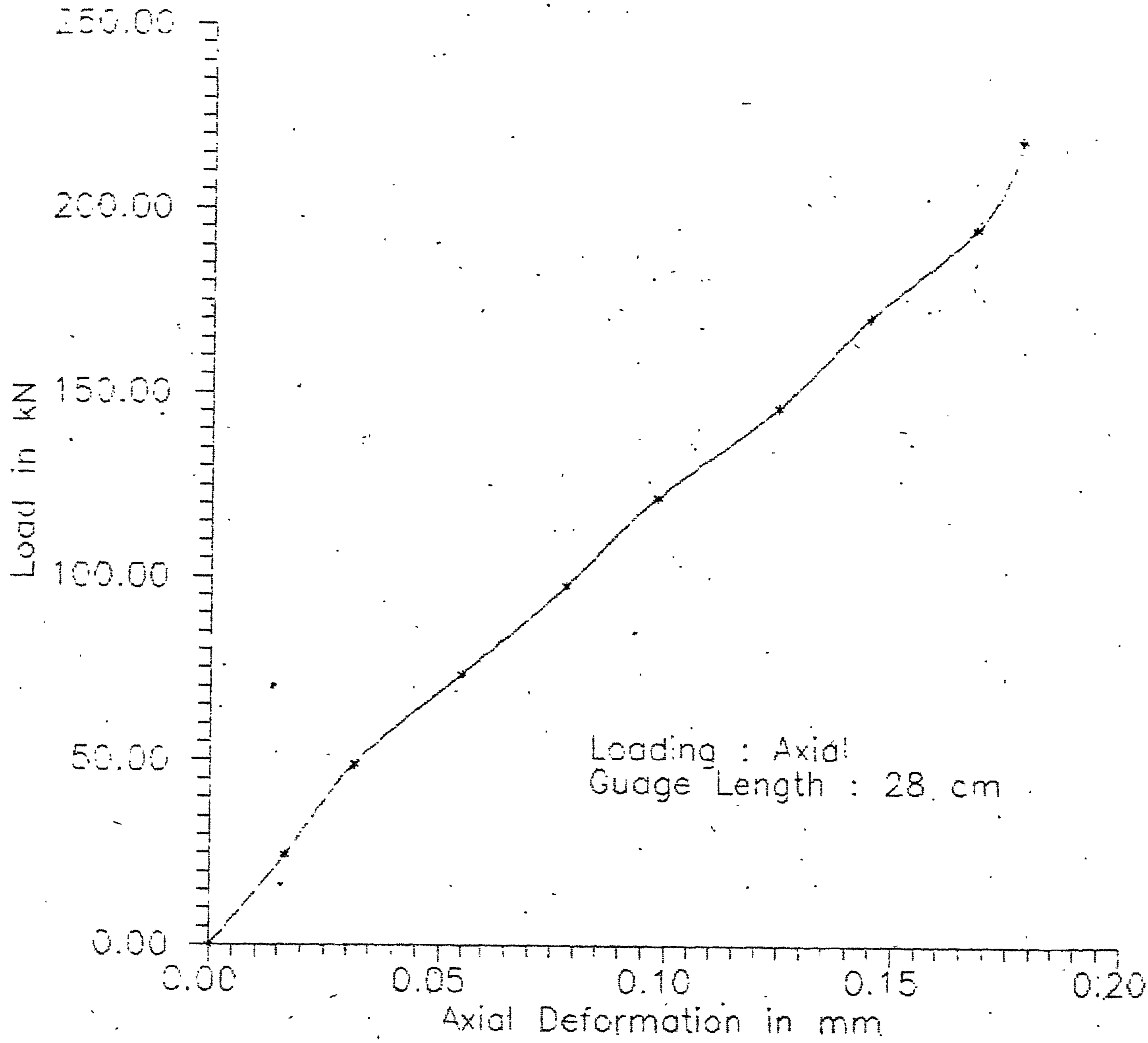
Load/Axial Deformation Curve, COLUMN-S2b

FIG. 4.18



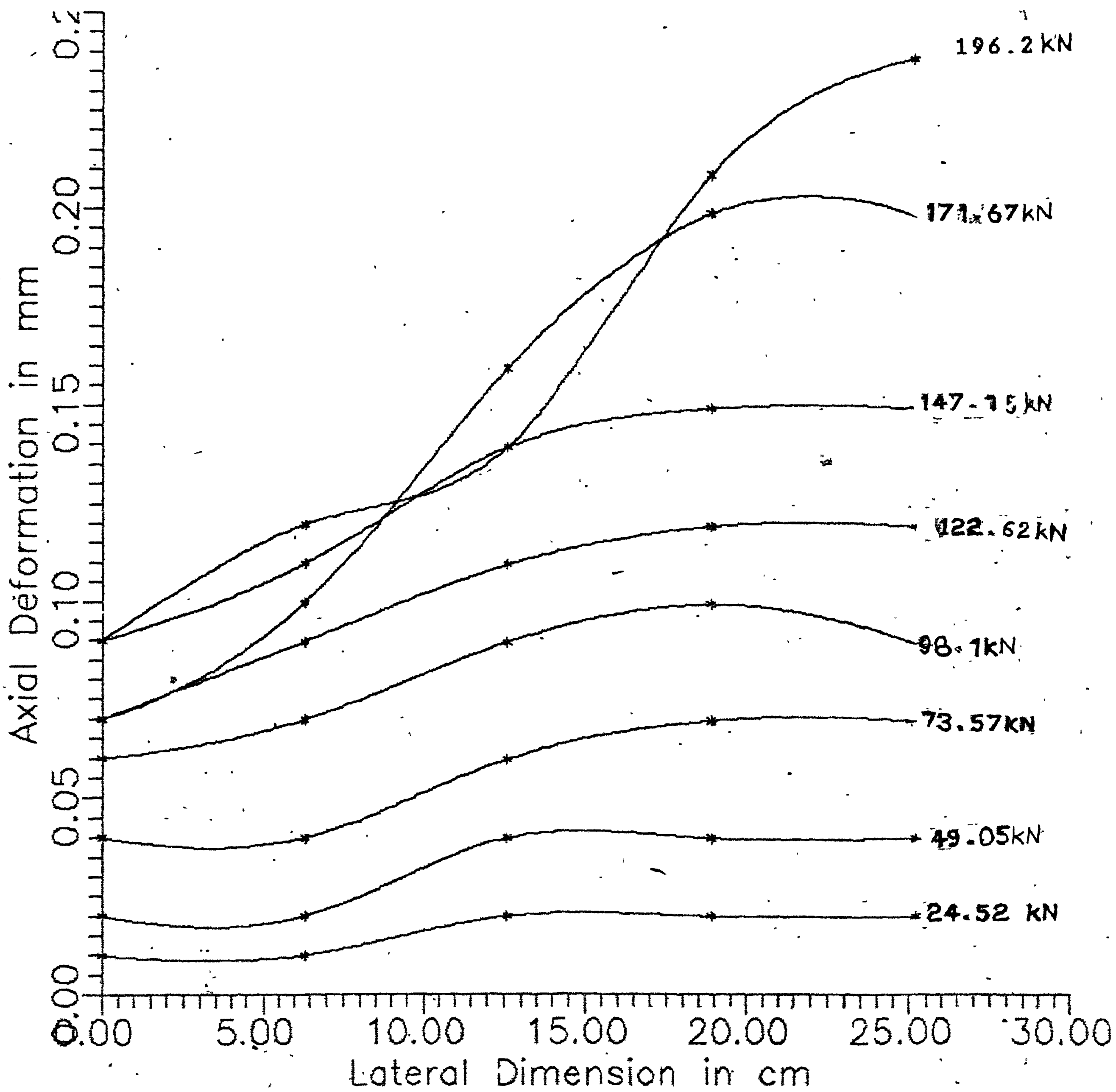
Axial Deformation/Lateral Dimension(at L.D.)
Curve, COLUMN-S2b

FIG. 4.19



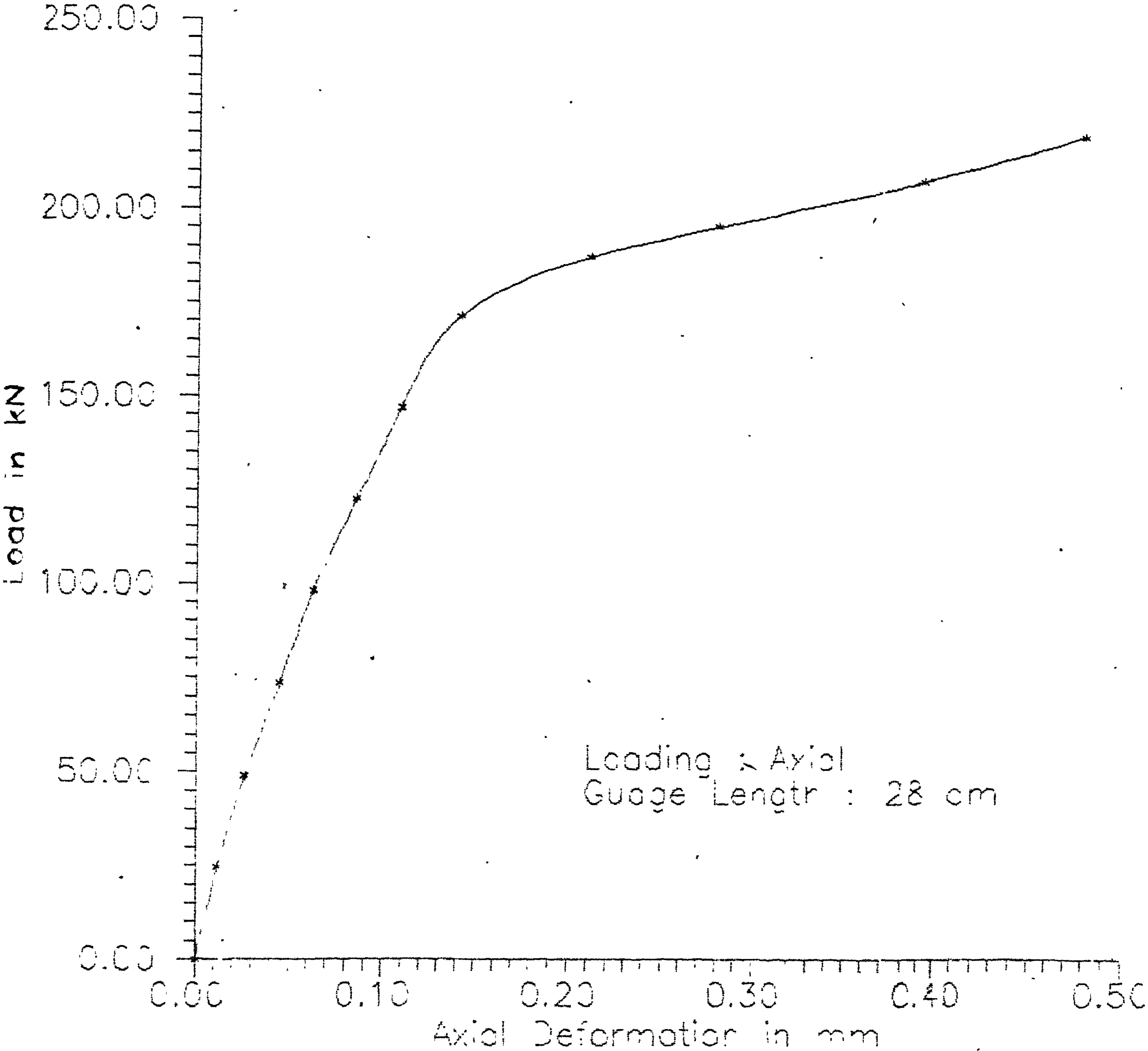
Load/Axial Deformation Curve.
COLUMN-S5

FIG. 4.20



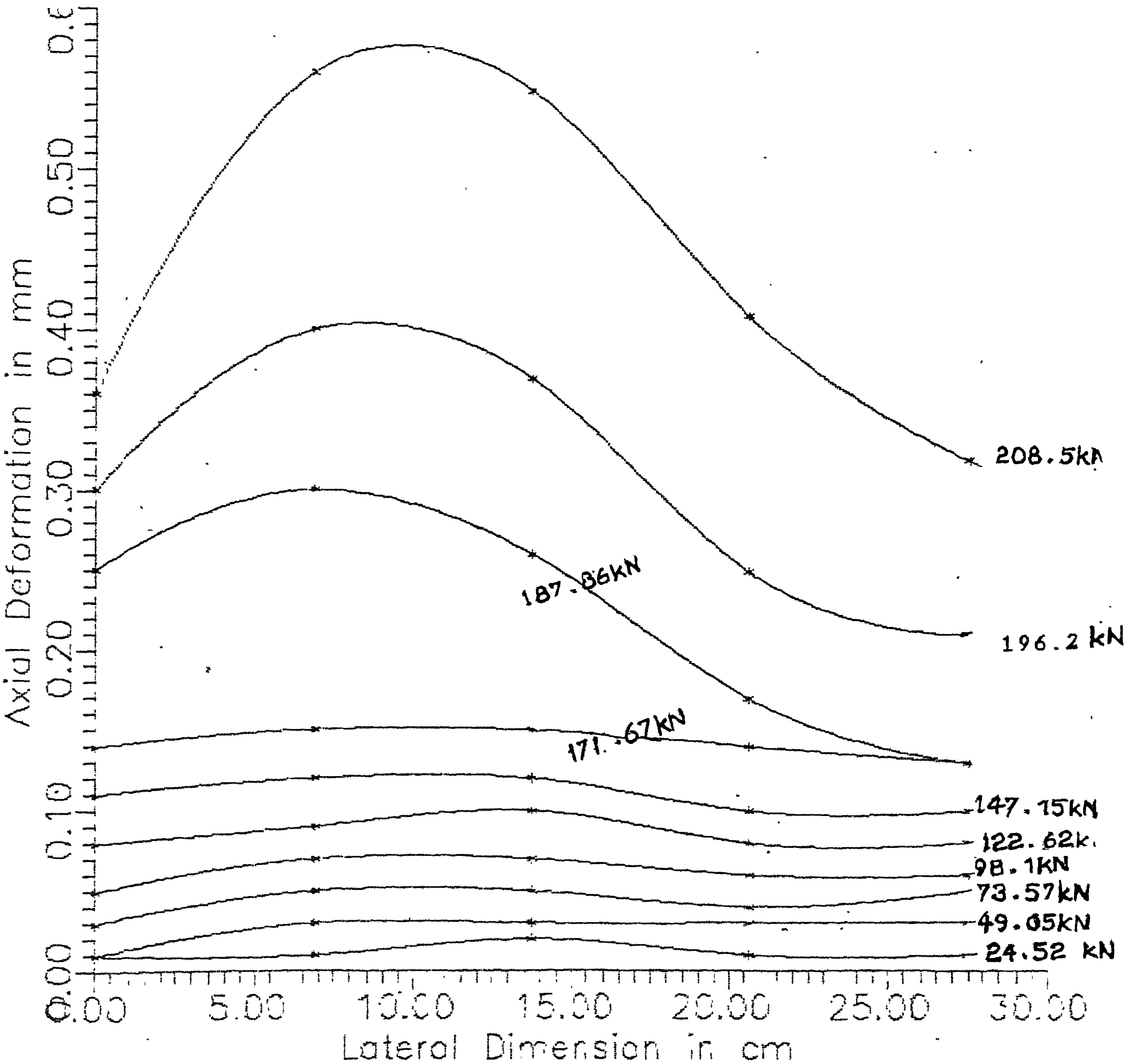
Axial Deformation/Lateral Dimension (at L.D.) Curve, COLUMN-S5

FIG. 4.21



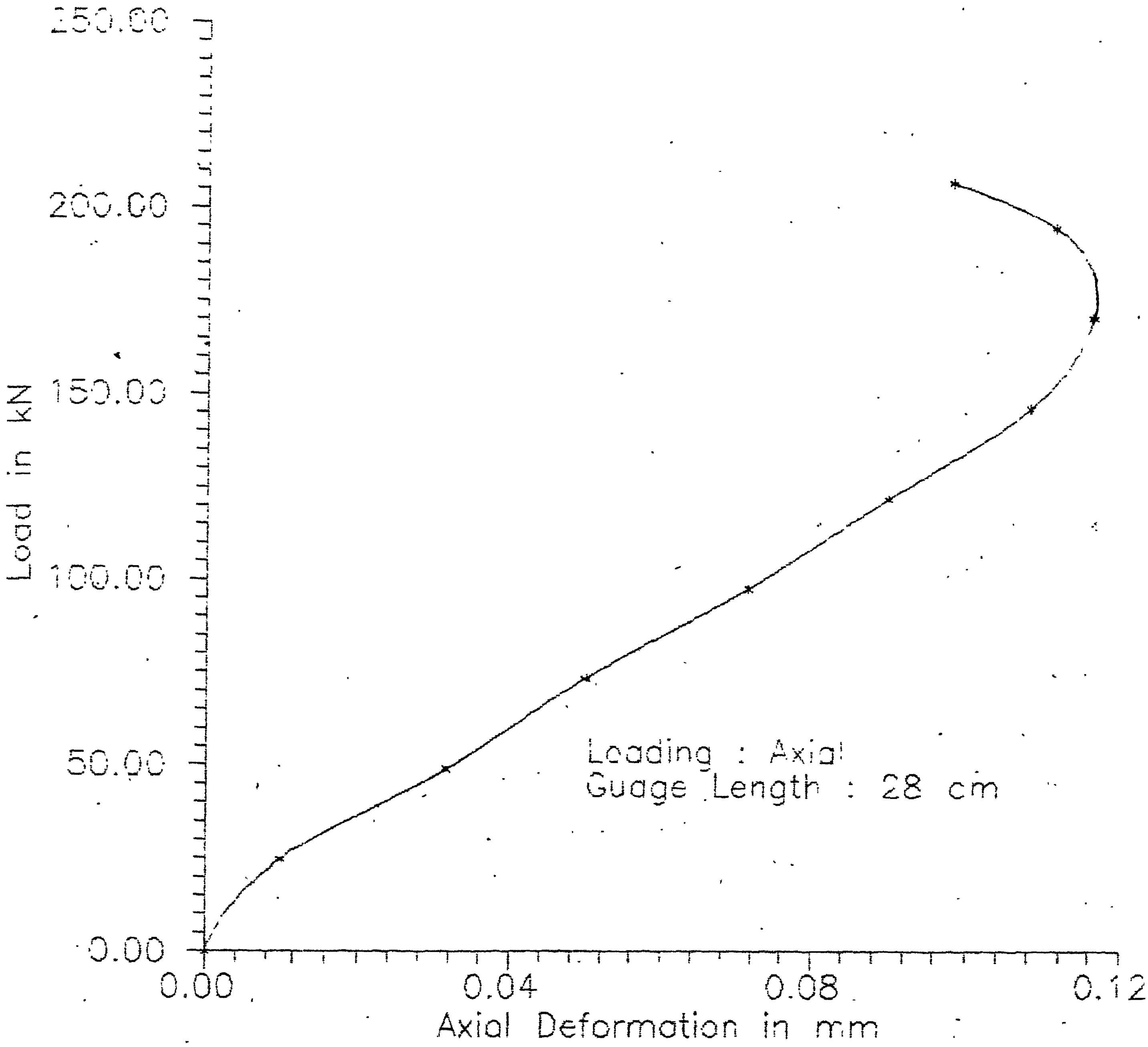
Load/Axial Deformation Curve, COLUMN-S4

FIG. 4.22



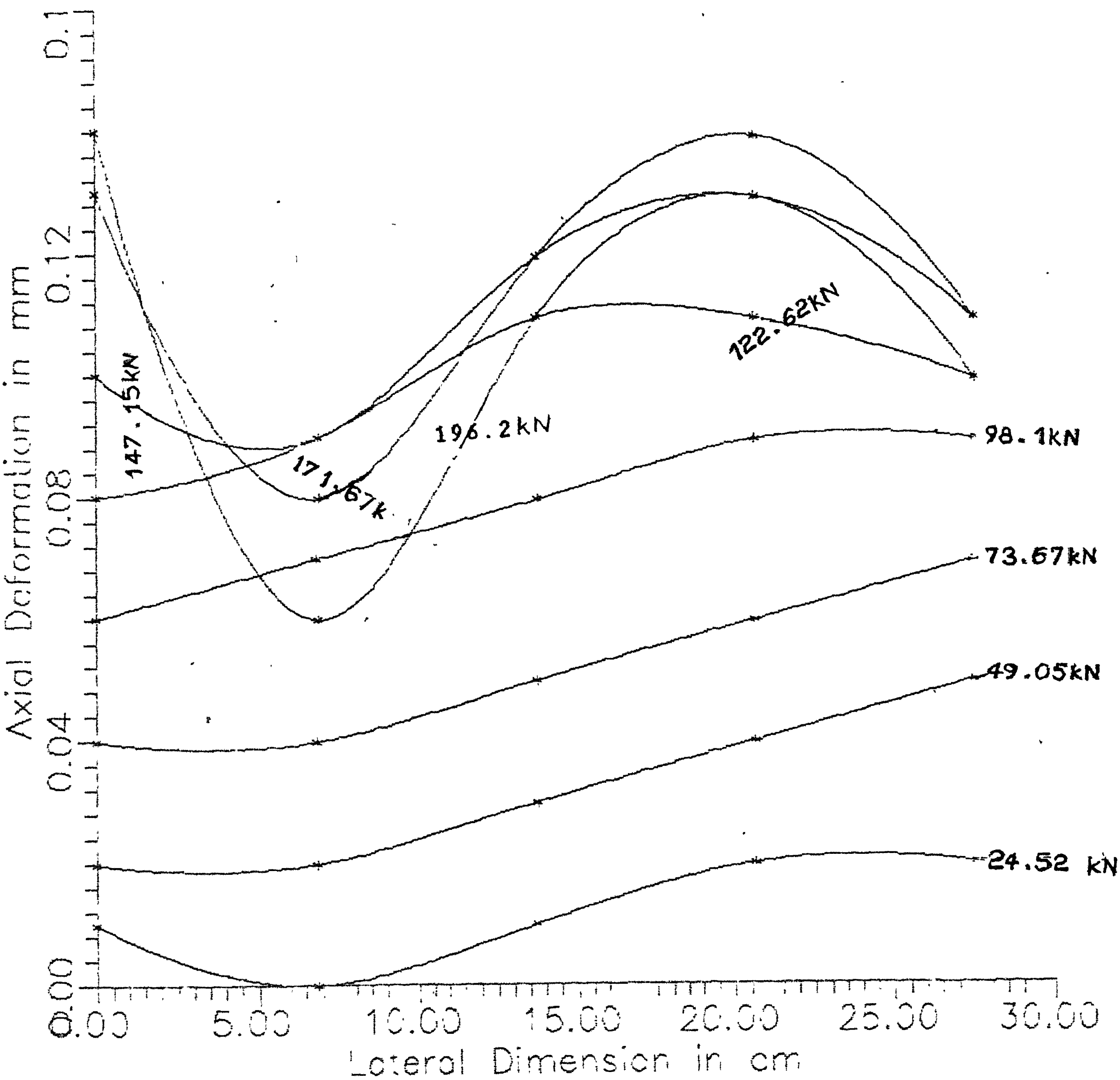
Axial Deformation/Lateral Dimension (at LD) Curve, COLUMN-S4

FIG. 4.23



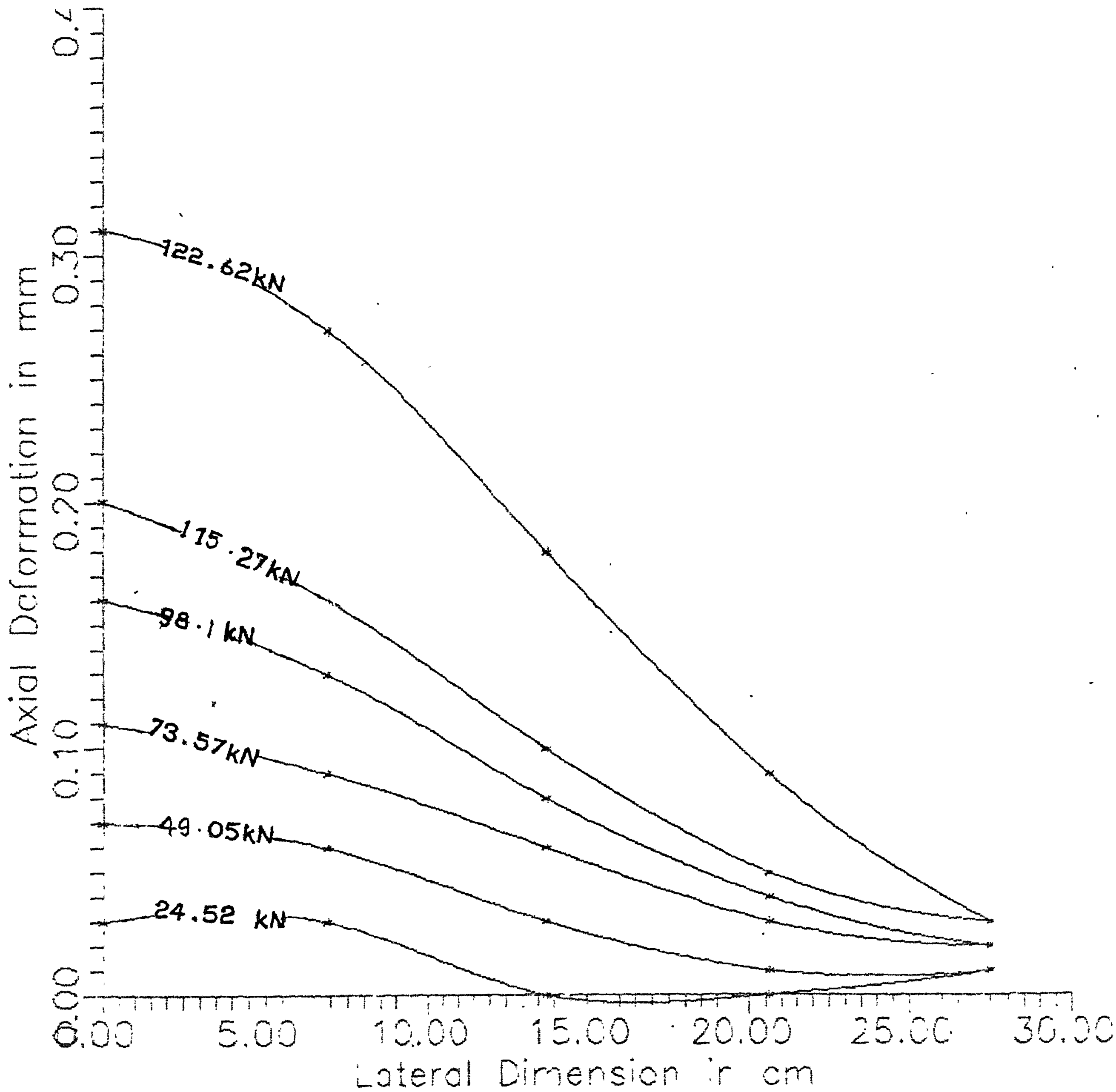
Load/Axial Deformation Curve, COLUMN-S7

FIG. 4.24



Axial Deformation/Lateral Dimension(at L.D.)
Curve, COLUMN-S7

FIG. 4.25



Axial Deformation/Lateral Dimension (at L.D)
Curve, COLUMN-S1a

FIG. 4.26

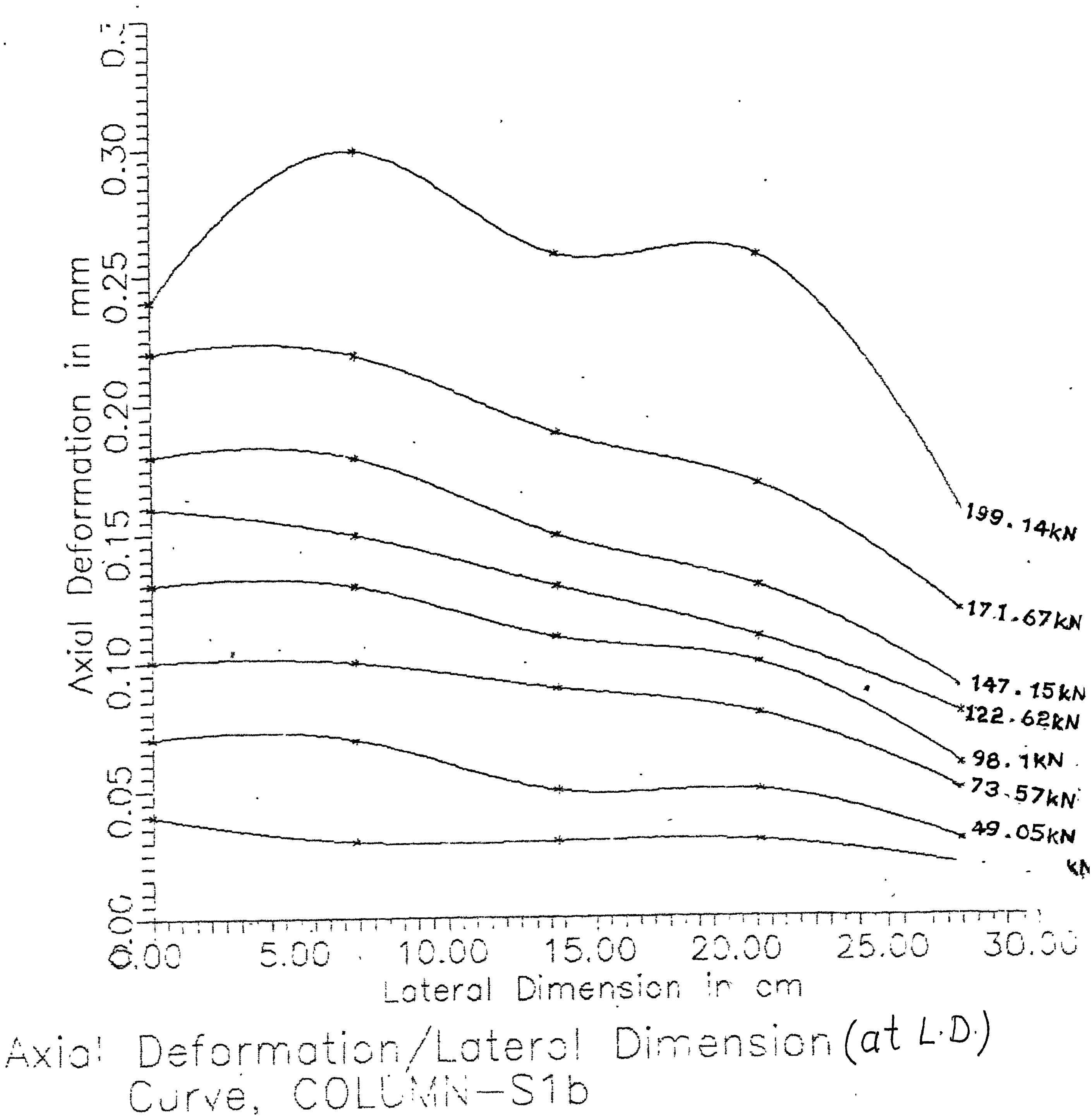
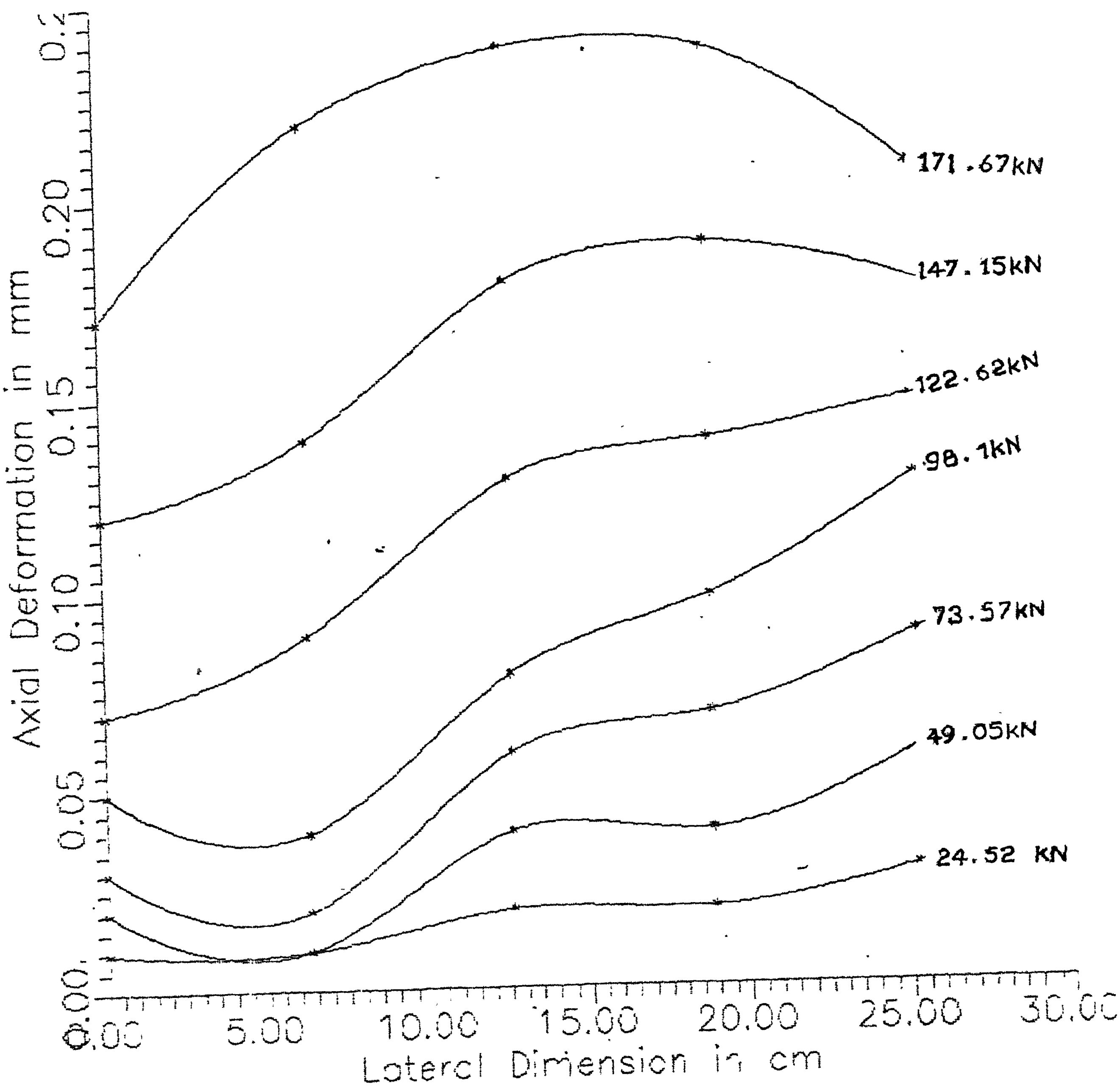
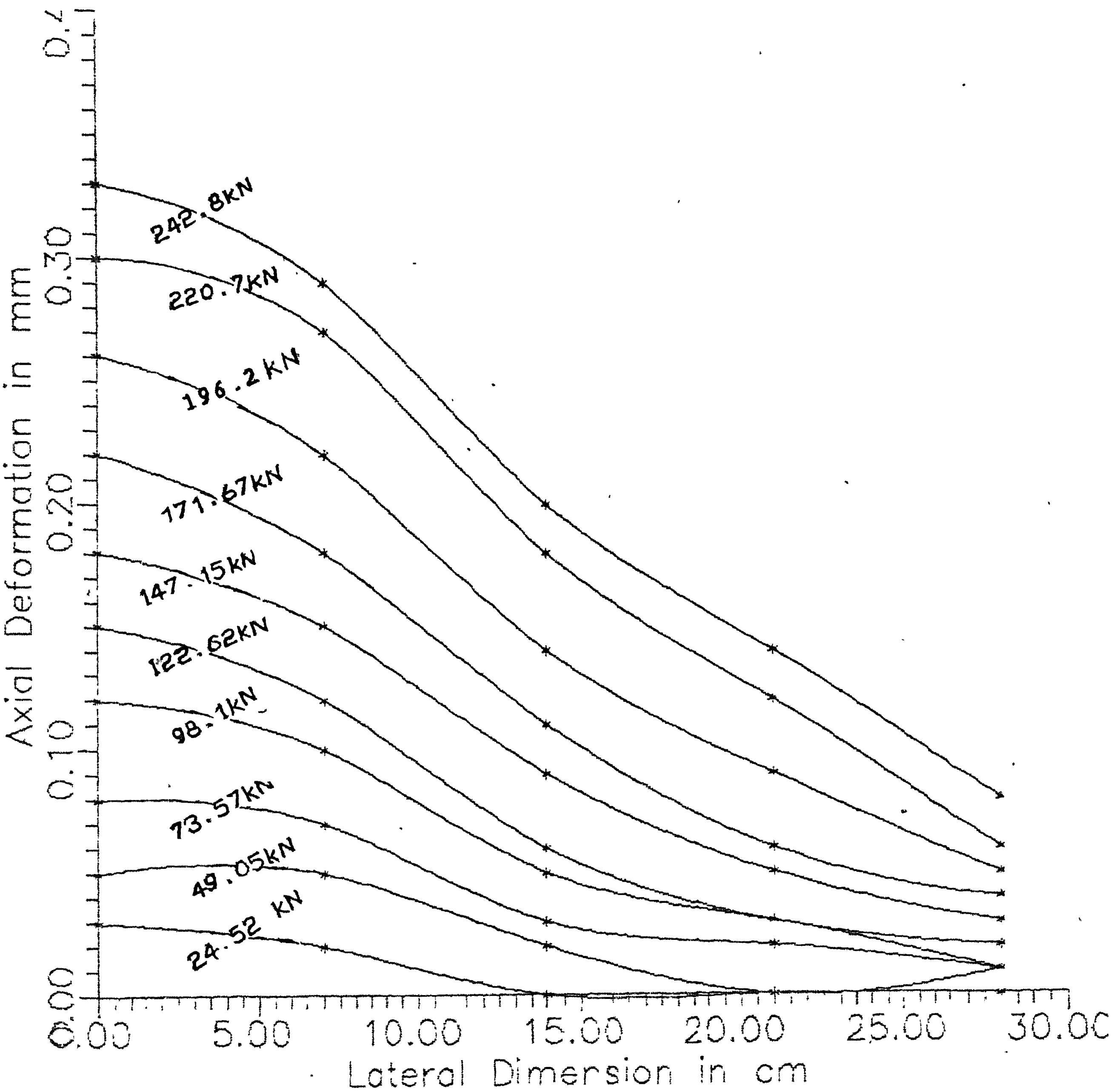

Axial Deformation in mm
0.00
0.05
0.10
0.15
0.20
0.25
0.30
0.00
5.00
10.00
15.00
20.00
25.00
30.00
Lateral Dimension in cm
199.14kN
171.67kN
147.15kN
122.62kN
98.1kN
73.57kN
49.05kN
Axial Deformation/Lateral Dimension (at L.D.)
Curve, COLUMN-S1b

FIG. 4.27



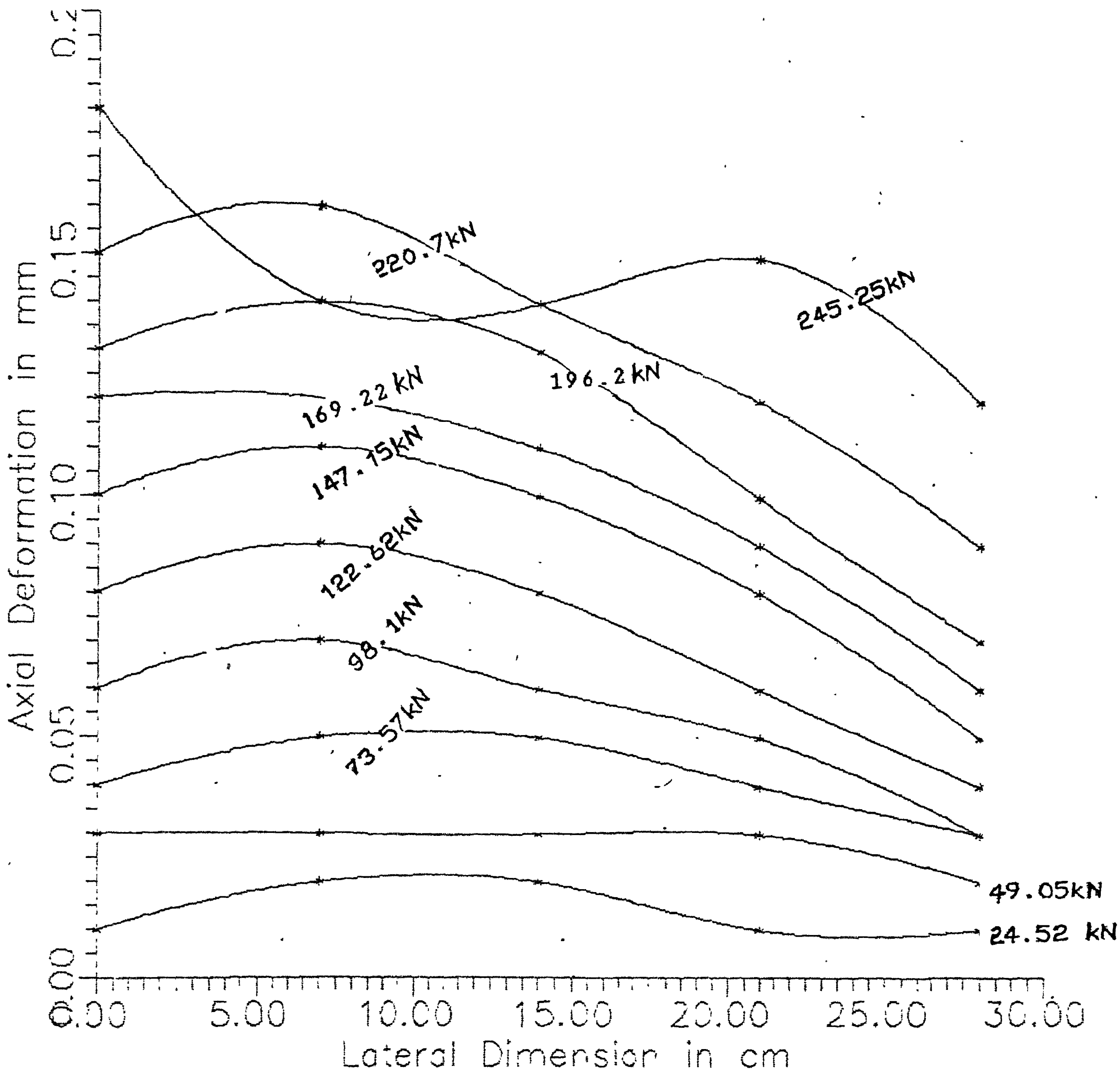
Axial Deformation/Lateral Dimension (at L.D.)
Curve, COLUMN—S8

FIG. 4.28



Axial Deformation/Lateral Dimension (at L.D.) Curve, COLUMN-S3

FIG. 4.29



Axial Deformation/Lateral Dimension (at L.D.)
Curve, COLUMN-S6

FIG. 4.30

length of failure zone increased. In all other axially loaded, unreinforced columns failure was localised upto not more than 30 to 35% of there lengths from top. Thus failure region/zone keeps on becoming more and more localised as length of column increases.

For eccentrically loaded, unreinforced columns, failure region was much less than that in case of axially loaded columns. The extent of failure region in this case varies with variation in eccentricity of load. The extent of failure region decreases with increase in eccentricity. For ferrocement encased columns, in both the cases i.e. axially as well as eccentrically loaded columns, the extent of failure region was found to be much more than that in case of unreinforced columns, for respective cases. Thus wire reinforcement avoids stress concentration near supports.

The mesh has not failed in any case. After failure and crushing of bricks, it was compressed and it lost the contact and became loose. It seems that due to thick and galvanized wires good bond could not develop. Good composite action, could not be achieved due to the same reason.

In all cases failure was initiated by vertical splitting of bricks. thus failure occurs when tensile strain in brick reaches its ultimate value. From graphs for axially loaded columns it is clear that strain is not uniform throughout the section and the trend of variation of strains through the section changes for different loadings.

This indicates the inhomogeneous behaviour of material inside the column. For eccentric loading also, the variation in axial deformation/strain is not linear but smooth curves are formed. In cases where dial gauge system has been fitted near the loading end, the strain variation is severe. It clearly shows that near the supports stress concentration takes place.

For columns without reinforcement and axial loading the load versus axial deformation curves remain straight line up to the point where strain reached the value 0.00045. The ultimate strain at which failure occurs was found to be 0.0005. For the columns having single layer of mesh reinforcement the proportional limit was reached when strain became 0.0004 and ultimate at which failure occurred, found to be 0.00045.

Average load carrying capacity of 2m columns having no reinforcement was found to be about 200 kN under axial loading. The corresponding value with single layer of reinforcement was found to be about 220 kN. Thus the increase in axial load carrying capacity was observed only to be 10% or so, which is much less than theoretical value. Variation of eccentricity on reinforced columns does not effect substantially the load carrying capacity. In case of eccentric loading the mesh is compressed in compression side and it bulges out. Thus it does not contribute to the strength of column.

Elastic constants in brickwork are difficult to establish and tend to vary with a number of factors. Considering the worst and

the best cases, we get from the equation (2.14) derived in Chapter II, the values presented in Table 4.14 for specimen columns having no reinforcement. The calculation of load capacities has been made by taking ultimate strain in brick to be equal to 0.001.

TABLE 4.14

Comparision of Actual and Theoretical Values of Average Load Capacity of Axially Loaded Unreinforced Columns.

Worst possible capacity (kN)	Best possible capacity (kN)	Actual/Experimental capacity (kN)
1550	7000	200

The actual average load capacity obtained from experiments has also been given in the table. It is very difficult to predict the elastic constants involvled in brick work. Moreover stress strain curve for mortar is not a straight line but is a curve. So mortar behaves more plastically than elastically.

In developing theory an ideal case has been taken where bricks as well as mortar are considered as an elastic homogeneous material. But that is not the actual case. The theoretical capacities are too large than actual ones and also, the range of theoretical values of capacity is also very large.

Considering all these facts it is clear that the theory presented for calculating load capacity for axially loaded column, could not provide satisfactory results. It should be refined and relooked.

CHAPTER V

CONCLUSIONS AND RECOMMENDATIONS

5.1 CONCLUSIONS

From the foregoing experimental and theoretical study the following conclusions may be drawn:

(i) Stress concentration takes place near supports. That is why local failure of columns occurs. So more reinforcement is necessary at supports than in the middle. It may also be possible to increase load bearing capacity by only providing reinforcement at the ends upto some length.

(ii) More the length, lesser was the zone of failure.

(iii) It is clear from failure behaviour of columns studied that good composite action could not be achieved due to thick and galvanized wire mesh.

(iv) Effect of wire mesh reinforcement in increasing load carrying capacity, in case of eccentric loading is much less as compared to that of axial loading.

(v) In case of axial loading, encasement increased the load carrying capacity by 10% or so.

(vi) For eccentrically loaded columns the effect of eccentricity does not very much influence the load carrying capacity.

(vii) The margin between experimental and theoretical capacity values is substantial which requires a relook into the modelling.

(viii) Encasement of columns increases the zone of failure, thus reduces stress concentration.

(ix) The wire mesh should be of higher gauge wires and spacing of wires should be less. Ungalvanized wire meshes are desirable, so that good composite action can develop. These flexible wire meshes can be wrapped around the column with ease.

(x) Horizontal overlap between different wire meshes should not be less than 15 cm.

(xi) To get good composite action the reinforcement should be uniformly provided throughout the thickness of plaster. Many layers of fine wire mesh should be provided instead of one thick wire mesh, having mortar in between them.

(xii) For axially loaded unreinforced columns, elastic behaviour of column ceases when strain is about 0.00045 and failure occurs at about 0.0005.

(xiii) For axially loaded single layered columns the strains at elastic limit and failure are 0.0004 and 0.00045 respectively.

5.2 SCOPE OF FURTHER STUDY

(i) By using smaller diameter and fine wire mesh and rich mortar more specimens should be cast and tested, so that good composite

action could be developed. It is expected that the load capacity will increase much more.

(ii) More reinforcement at supports than in the middle portion should be provided.

(iii) Varying amounts of reinforcement over different regions along the length besides type of wire mesh may also be studied. It should be seen that failure occurs by splitting of column along the whole length for axial loading. The simultaneous failure of bricks and mesh should be attained.

(iv) Ferrocement encased columns should be tested under dynamic loading

(v) The brick columns may also be reinforced by providing angles along the edges besides fine wire mesh.

(vi) Some specimens should be made and tested by providing multilayered mesh cover at the top and bottom of the columns.

(vii) Experiments could be conducted on columns having various lengths and sections.

(viii) The upper limit for the reinforcement should be established.

(ix) Specimen should be made by providing uniform fine wired reinforcement throughout the thickness and tested.

(x) Theory of failure of axially loaded columns should be relooked and refined.

(xi) Correction factors for various degrees of slenderness should be established.

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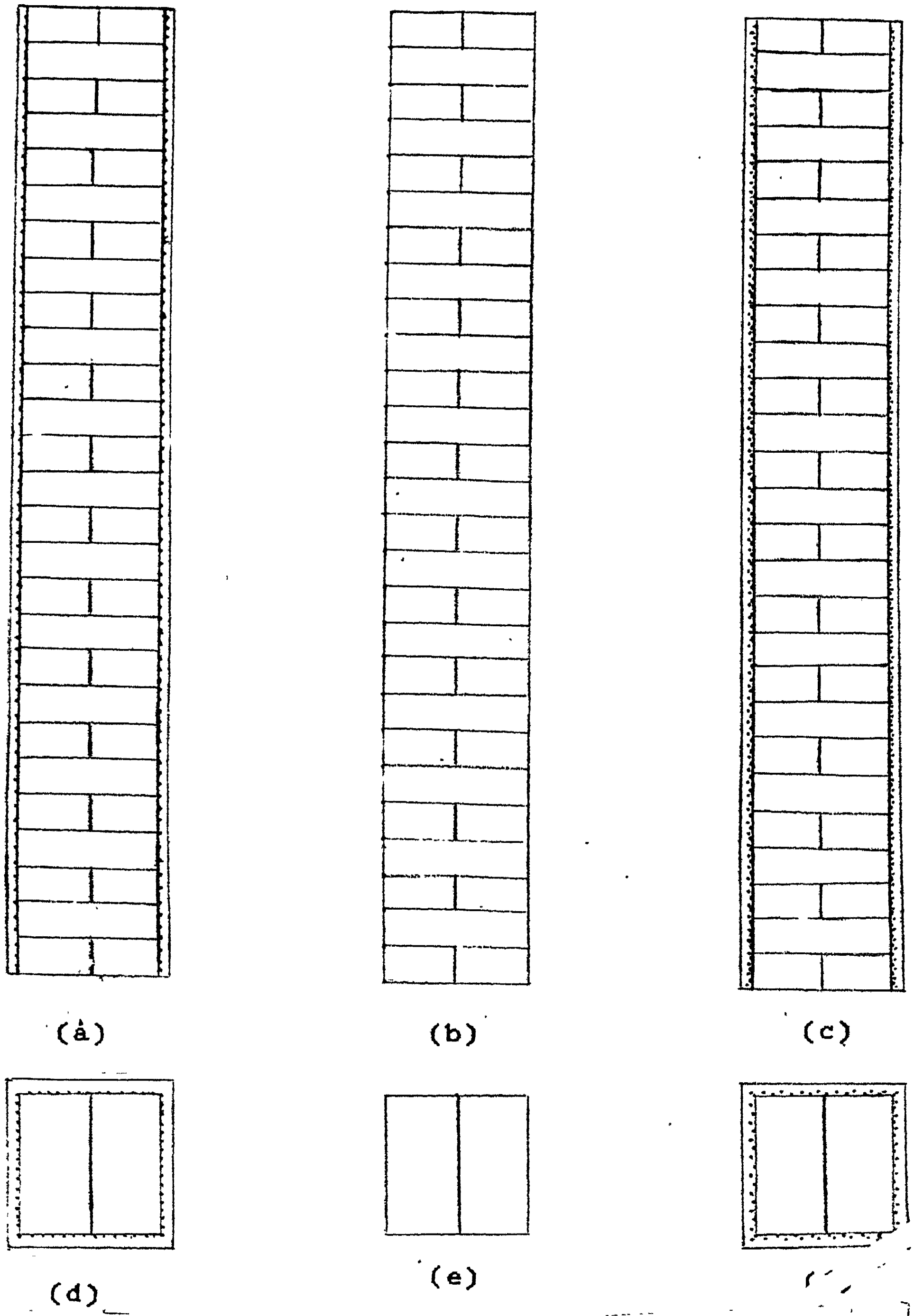

Fig.Showing details of specimen columns

(a) Longitudinal section through one layered encased column
(b) Longitudinal section through masonry core
(c) Longitudinal section through two layered encased column
(d) Transverse section through one layered encased column
(e) Transverse section through masonry core
(f) Transverse section through two layered encased column

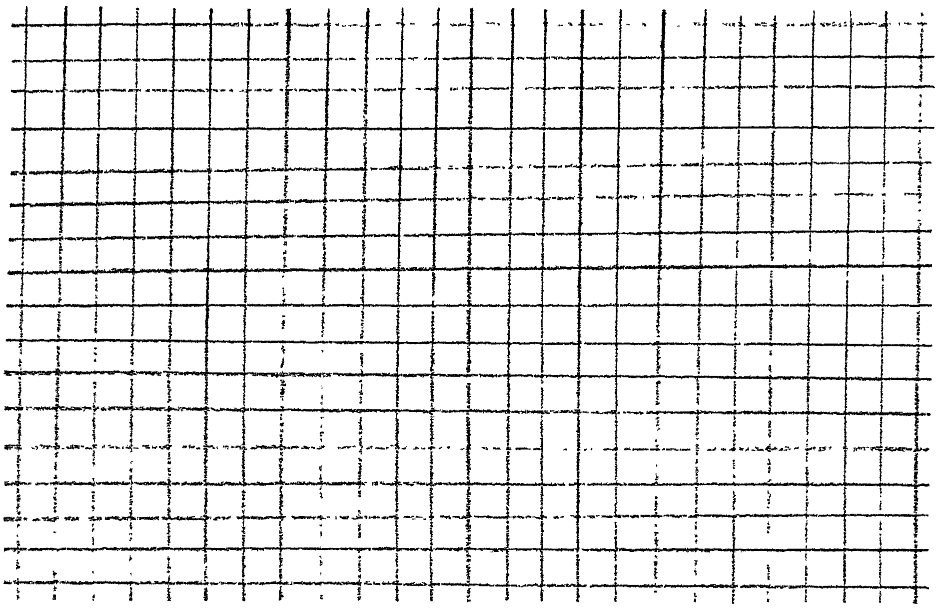

Fig.Showing wire mesh used for reinforcement

Woven square wire mesh
Dia of wire = 1mm
Spacing of wires = 1cm